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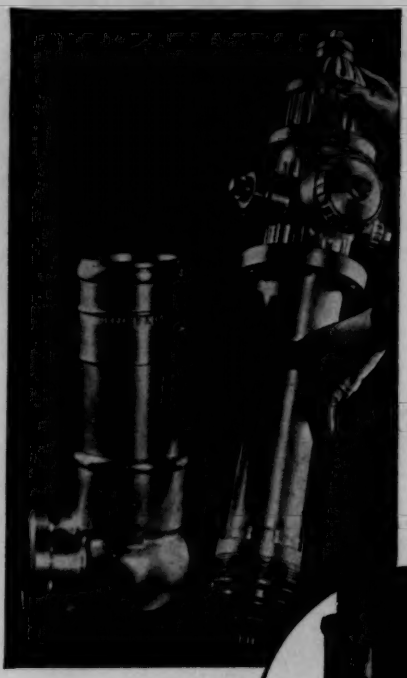

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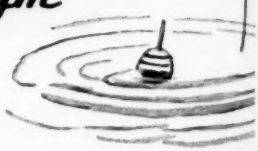

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Vol. 31 November, 1939 No. 11

Methods of Calculating Water-Hammer Pressures

By F. M. Dawson and A. A. Kalinske

WITHIN the last decade our knowledge regarding the calculation of pressures caused by water hammer or surges in pipe lines has increased tremendously. The general theory has been known for decades; however, the practical application of this theory to the determination of water-hammer pressures, except for the simplest of conditions, was not developed very extensively until quite recently. The reasons for this were primarily the mathematical difficulties that were encountered and the lack of reliable experimental facts. Due to the impetus given the study of water-hammer problems by the two Water-Hammer Symposiums held in this country in 1933 and 1937 under the sponsorship of the A. S. M. E., A. S. C. E., and A. W. W. A., considerable information relating to the calculation of water-hammer pressure has been made available. However, much of this information is presented in a condensed form, and is couched in mathematical terms of such nature that the practical use of this information by water works engineers and designers has not been extensive.

It is the intent of the authors to attempt to present the various methods available for the calculation of pressures caused by water hammer in a more simplified manner. The aim of this paper is to

A paper presented on June 13, 1939 at the Atlantic City Convention by F. M. Dawson, Dean, College of Engineering, University of Iowa; and by A. A. Kalinske, Research Engineer, University of Iowa, Iowa City, Iowa.

make available to practicing engineers in the water works field methods for calculating water-hammer pressures, not only under simplified conditions, but also under the various more complicated conditions encountered in the field. The various theories presented, and the bases for the different methods of calculation, are not original with the authors; they are taken from the many papers on this subject, of which the more important in the English language are listed at the end of this paper.

General Theory

Before beginning the discussion of the calculation of water-hammer pressures, it would be well to review the accepted concept of water hammer and how these pressures act. Taking a simple conduit with a valve at one end and a tank, reservoir, or a much larger conduit at the other, the momentum-impulse analysis shows that the excess pressure created by an instantaneous closure of the valve where the velocity was V_0 is equal in feet of the liquid flowing to:

$$h = \frac{a}{g} V_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where a is the velocity of the pressure wave and g is the gravity constant. This excess pressure wave travels up the conduit towards the reservoir, thus placing the whole conduit under an excess pressure. After this, the water begins to flow back out of the conduit and the pressure gradually drops to normal along the whole length of the conduit, starting at the reservoir and proceeding toward the valve at a velocity of a feet per second. If L is the length of the conduit, then at an interval of time of $2L/a$ seconds after valve closure, the whole conduit is again under a normal pressure. However, at this instant, due to the "spring" action of the elastic walls of the pipe or conduit and the elasticity of the water, a negative wave starts out from the valve equal in magnitude to the pressure wave. That is, the pressure now goes below normal as much as it previously had gone above normal. This negative wave proceeds up the conduit and at a time of $3L/a$ seconds after closure, the whole conduit is under the sub-normal pressure. The water from the reservoir then starts to flow back into the conduit and the pressure rises to normal at the gate at a time of $4L/a$ seconds after closure. This is the complete wave cycle. Theoretically, this wave cycle would

be repeated again and again; actually, however, it is damped rapidly due to the energy of the wave being dissipated by friction, i.e. heat.

Figure 1 shows a graphical picture of this wave cycle and how it proceeds along a conduit. The theoretical forms of the waves for the gate end, quarter-point, and midpoint of the conduit are also indicated.

From this analysis it becomes apparent that in order to obtain the maximum water-hammer pressure, as given by Equation 1, the valve should be closed at least in the time $2L/a$. After a time $2L/a$, the part of the pressure wave which was started up the conduit by the first part of the valve closure will have returned and will be reflected as a sub-normal wave, thus decreasing the pressure rise at the gate.

The calculation of the water-hammer pressure for a reduction in velocity occurring within the time $2L/a$ is quite simple, and is according to Equation 1. This general concept was first definitely proved and experimentally verified by Joukovsky (1). The wave velocity a can be readily calculated if the conduit and liquid characteristics are known. The formula for a in ft. per sec. is:

$$a = \frac{12}{\sqrt{\frac{w}{g}\left(\frac{1}{K} + \frac{D}{SE}\right)}} \qquad (2)$$

w = Unit weight of the liquid, lb. per cu.ft.
K = Bulk modulus of elasticity of liquid, about 300,000 lb. per sq.in. for water.
D = Pipe diameter in inches.
S = Pipe wall thickness in inches.
E = Linear modulus of elasticity of pipe material in lb. per sq.in.
g = 32.2 ft. per sec. per sec.

The proof of this formula is given in most hydraulics textbooks.

To calculate the maximum water-hammer pressure when valve closure takes place in a time longer than $2L/a$, it is necessary to divide the total time of closure into intervals of $2L/a$, or fractional parts of $2L/a$, and to follow through the various pressure and sub-normal waves in detail in order that they may be combined correctly. The pressure rise at the beginning of the second $2L/a$ interval, for instance, is altered by the sub-normal wave starting at the gate due to the reflection of the super-normal pressure wave created at the start of the first $2L/a$ interval. Suppose a certain part of the velocity is

reduced by $(\Delta V)_1$ during the first time interval, $2L/a$. For simplicity it can be assumed that this change in velocity occurred instantaneously at the beginning of the time interval. During the second interval, a decrease in velocity of $(\Delta V)_2$ occurs, which for calculation purposes can be assumed to occur instantaneously at the beginning of the second interval. The pressure rise during the first interval was $h_1 = \frac{a}{g}(\Delta V)_1$ and the pressure rise due to the velocity change, $(\Delta V)_2$, is $h_2 = \frac{a}{g}(\Delta V)_2$. However, the total pressure rise at the beginning of the second $2L/a$ time interval is not the sum of these two values, because at the end of the first interval of $2L/a$ seconds, the pressure, h_1, has been reflected and starts out at the valve as a subnormal wave of magnitude h_1. Therefore the pressure during the second interval is $(h_1 + h_2) - 2h_1$.

In order to calculate the actual values of $(\Delta V)_1$ and $(\Delta V)_2$ for the selected time interval, it is necessary to know how the valve is closed, and the relation between the velocity in the pipe, the head at the valve, and the discharge coefficient of the valve. Let $V_0 = B_0\sqrt{H_0}$ be the relation at the valve between the initial head in the pipe, H_0, and the initial velocity in the pipe, V_0. The number B_0 is the ratio of the area of the valve opening to the area of the pipe multiplied by $\sqrt{2g}$ times the coefficient of discharge of the valve opening. For uniform gate closure, the relation at any time t between B_0 and the value of B at that time is: $B = (1 - \frac{t}{T})B_0$, where T is the total time of closure. During the time t, the pressure in the pipe has risen by the amount h_t so that the total head at the valve, H_t, equals $H_0 + h_t$. The expression for the velocity in the pipe at the end of t is then:

$$V_t = \left(1 - \frac{t}{T}\right) B_0\sqrt{H_0 + h_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The value of h_t, the total water-hammer pressure rise in time t, is determined by adding algebraically all the positive and negative pressure waves that are traveling in the pipe at the point under consideration. This method of analysis was first presented by Gibson (2), and is called the method of "arithmetic integration." A simple example will make clear the method of calculation.

The data for this problem are as follows: $L = 1000$ ft., $V_0 = 10$ ft.

per sec., $a = 4{,}000$ ft. per sec., total time of valve closure is 2 sec., $H_0 = 100$ ft. The problem will be solved by computing (Δh) for each half interval, or since $2L/a$ is $\frac{1}{2}$ sec., for each $\frac{1}{4}$ sec. In table 1 the calculated values are tabulated. In computing the value of h_t or $\Sigma\Delta h$, it is best to sketch up a wave pattern for the period of time selected (a half interval in this case); in this way it can be readily seen whether a certain Δh should be added or subtracted. The value of h_t at the end of the first interval is, of course, the sum of the values of (Δh) calculated for the first two half intervals. At

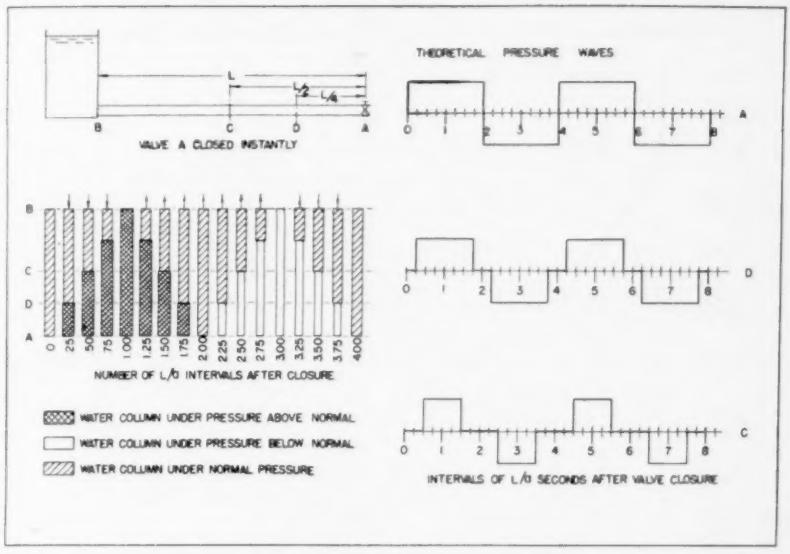

FIG. 1. Graphical Representation of Water-Hammer Waves in a Simple Conduit for Instantaneous Valve Closure

the end of $1\frac{1}{2}$ intervals, h_t is equal to the sum of the values of Δh for the first three half intervals minus twice the value of Δh for the first half interval. The reason twice the value of the subtractive Δh is taken is because the pressure drops from a positive value to an equal negative value when it returns as a negative reflected wave. The calculation for h_t at the other intervals is similar. The total maximum pressure occurs at the end of the closure and is equal to 430 ft. or 330 ft. above static.

From this example, it is apparent that the general relation between the water-hammer pressure at the end of any time interval n, (where

an interval is $2L/a$ seconds), and the pressure at the end of the previous interval is as follows:

$$h_n - h_{n-1} = \frac{a}{g}(V_{n-1} - V_n) - 2h_{n-1} \quad \ldots\ldots\ldots\ldots (4)$$

By adding equations of this type, we obtain:

$$h_n = \frac{a}{g}(V_0 - V_n) - 2(h_1 + h_2 \cdots + h_{n-1}) \quad \ldots\ldots\ldots (4a)$$

TABLE 1

Water-Hammer Computations by Method of Arithmetic Integration

TIME	INTERVAL	B_t	H_t	V_t AND ΔV	Δh	h_t
sec.						
0.0	0	1.000	100.0	10.00		0.0
				0.21	25.4	
.25	½	0.875	125.4	9.79		25.4
				0.28	35.2	
.50	1	0.750	160.6	9.51		60.6
				0.71	88.2	
.75	1½	0.625	198.0	8.80		98.0
				0.95	118.0	
1.00	2	0.500	245.6	7.85		145.6
				1.41	175.0	
1.25	2½	0.375	295.0	6.44		195.0
				1.77	220.0	
1.50	3	0.250	349.0	4.67		249.0
				2.18	271.0	
1.75	3½	0.125	395.0	2.49		295.0
				2.49	309.0	
2.00	4	0.0	430.0	0.0		330.0

By combining Equation 4a with Equation 3, a general equation can be obtained for calculating the pressure at any time after closure. However, the equation is quite complicated and not very convenient to solve by algebraic means. Charts for solving this equation are given by Allievi (3). One of these charts is reproduced in fig. 2. For our example given previously, ρ, the pipe-line constant, is 6.2 and r, the time of closure in number of $2L/a$ intervals, is 4, and the chart gives the value of 4.3 for $\frac{H_0 + h}{H_0}$. Since H_0 was 100, the water-hammer pressure, h_t, is 330 ft. which checks the result obtained by the method of arithmetic integration. The dotted curved lines

in this chart indicate the interval during which the maximum pressure occurs; they are numbered from left to right starting with 1, and the last curve on the right is 17. In our example, the maximum pressure occurs almost at the end of 4 intervals, or at the end of the valve closure.

Principles of Graphical Method of Calculation

In general, the calculation of water-hammer pressures for gradual gate closure involves the solution of two simultaneous equations: one giving the relation between the water-hammer pressure rise

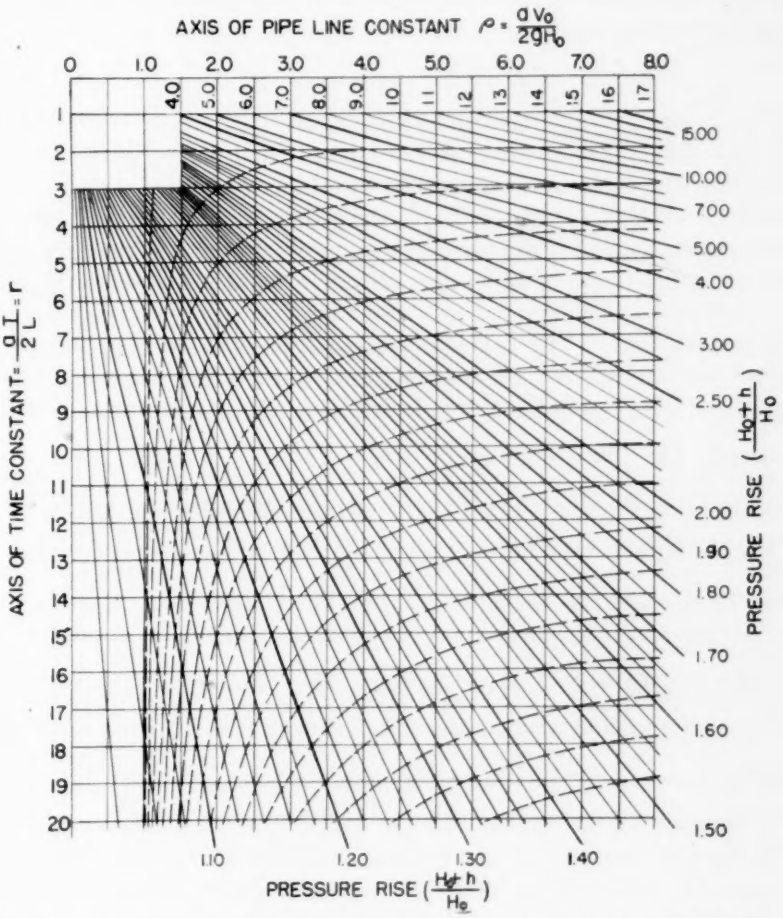

FIG. 2. Allievi Chart for Solving Water-Hammer Problems in Simple Conduits for Uniform Gate Closure

and the extinguished velocity, and the other the relation between the head at the gate, the velocity in the pipe, and the discharge characteristics of the gate. From Equation 4, the pressure rise due to water hammer at the end of any time interval $2L/a$ can be written thus:

$$h_n = (H_n - H_0) = \frac{a}{g}(V_{n-1} - V_n) - (H_{n-1} - H_0) \quad \ldots\ldots (5)$$

where n represents the number of the interval that has elapsed since closure began, and H_n is the total pressure, that is, $H_n = H_0 + h_n$. The other equation is that given as Equation 3 and which can be written as:

$$V_n = B_n\sqrt{H_n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

If the mode of valve closure is known, then B_n can be calculated at any time. The solution of Equations 5 and 6 graphically is as follows: The term B_n is calculated for the beginning of closure, at the end of the first interval $2L/a$, the second interval, etc. With V as the abscissa and H as ordinate, a series of parabolas as represented by Equation 6 can be plotted as shown in fig. 3. The value of B_0 in this case is 1.0, and we are assuming linear valve closure with respect to time. The problem being solved is the one previously presented for solution by the arithmetic integration method. The pressure, H_1, is obtained by determining the intersection of the straight line whose equation is $(H_1 - H_0) = \frac{a}{g}(V_0 - V_1)$ and the curve $V_1 = B_1\sqrt{H_1}$. The straight line is drawn through the point (V_0, H_0) with a slope of $-\frac{a}{g}$. The point of intersection determines the pressure, H_1, at the end of the first interval.

The pressure, H_2, at the end of the second interval is determined by graphically solving the equations $(H_2 - H_0) = \frac{a}{g}(V_1 - V_2) - (H_1 - H_0)$ and $V_2 = B_2\sqrt{H_2}$. The straight line will have again a slope of $-\frac{a}{g}$, and it will pass through the point M_1 which is below the H_0 axis by the amount $(H_1 - H_0)$. The point, H_2, gives the pressure at the end of the second interval. The construction is continued till the valve is closed, or to any desired partial closure. In this case, as in all our analyses so far, friction has been neglected.

When it is neglected, the results obtained for water-hammer pressure are greater than actually would be obtained. If the friction loss is a considerable part of the total static head, it should be considered.

The usual method by which friction is considered in the graphical method of solution is to assume it as concentrated at some one point in the conduit. This, of course, is not theoretically correct; however, practically, results are obtained which are roughly indicative of the effect of friction loss on water-hammer pressure. Friction loss tends to reduce the water-hammer pressure. The problem worked

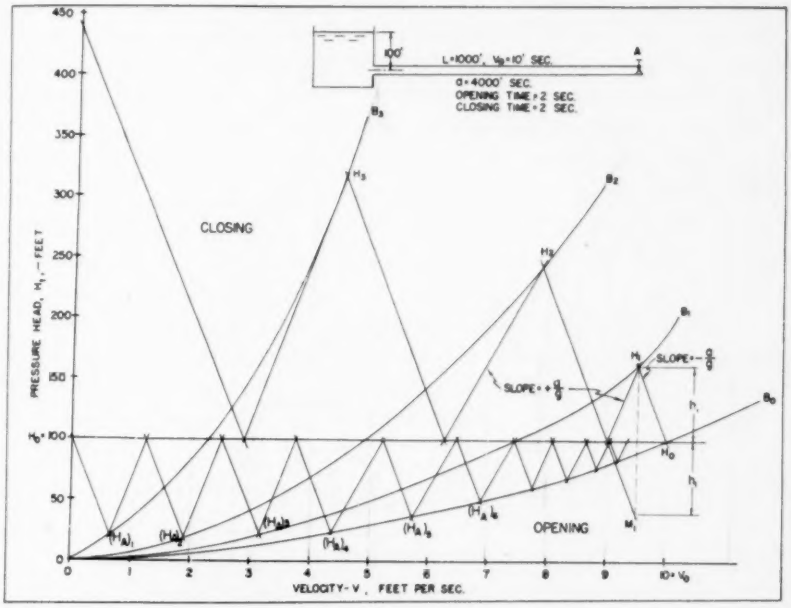

FIG. 3. Graphical Solution of Water-Hammer Problems for a Simple Conduit for Gradual Gate Closing and Opening

out in fig. 3 without considering friction is reworked in fig. 4 considering a total friction loss of 50 ft. in the entire conduit when the velocity is 10 ft. per sec. It is considered that the friction loss is concentrated at the gate end of the conduit. The distance between the curved line below the horizontal axis and this axis is the total loss due to friction for various velocities of flow in the conduit. The graphical construction proceeds as follows:

The intersection of the friction loss curve with the curve giving the initial relation between head and discharge at the gate represents

the initial velocity in the conduit under the assumed conditions. The initial velocity is 8.4 ft. per sec. as indicated by the location of point A_0' in fig. 4. Point A_0'' is located on the H_0 line just above A_0'. The line, $A_0''H_1'$, is drawn at a slope of $-\frac{a}{g}$. The point, H_1, is located then on the B_1 curve so that the vertical distance $H_1'H_1$ is equal to the friction loss at that velocity. Proceeding to the determination of H_2, a line of slope $+\frac{a}{g}$ is drawn through H_1' intersecting the H_0

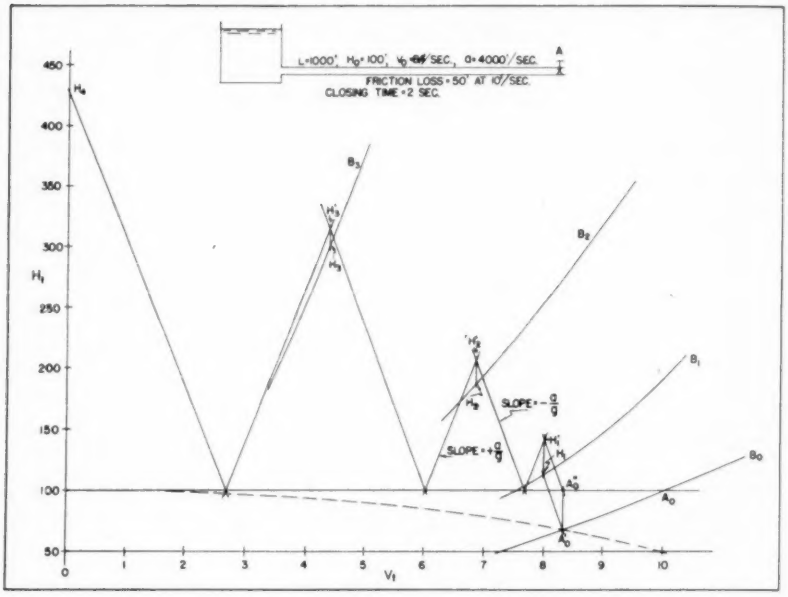

FIG. 4. Graphical Solution for Water-Hammer Pressure in a Simple Conduit for Gradual Gate Closure Considering Friction in Pipe

axis. At the point of intersection, a line of slope $-\frac{a}{g}$ is drawn, and H_2 is then located on the B_2 curve as was previously described. Note that in this case the final water-hammer pressure at A is not much different from the value obtained when friction was not considered.

For the case of valve opening, the graphical solution is quite similar to that for valve closing and is shown in fig. 3. The conditions of the illustrative problem are the same as for the case of valve closure,

except that now the valve is opened in 2 seconds, the initial velocity is zero and the final velocity is 10 feet per second. The graphical construction is similar to that for the valve closure shown in fig. 3 and no detailed explanation is necessary. Note that now the pressure after 1 sec., $(H_A)_1$, lies on the B_3 head-discharge curve, and that $(H_A)_2$ lies on the B_2 curve, and that $(H_A)_3$ lies on the B_1 curve, etc. After 2 seconds, the pressures, of course, all lie on the B_0 curve. The final velocity and pressure condition are not obtained till a considerable time elapses after the time taken to open the valve.

Compound Pipes

It is to be noted that the graphical method of solution just described can be conveniently used for determining pressure rise only at the gate end of a conduit, and also the pressure must be determined in steps of $2L/a$ sec. There is a slightly different graphical method of calculation which has gained considerable importance within the last few years because of its wide applicability to all conditions. The method was developed by Schnyder of Switzerland and Bergeron of France, and was presented to American engineers by Angus (4).

The development of the equations used in this graphical method need not be gone into as it is simply a process of mathematically manipulating the fundamental equations of water-hammer pressure. The equations used are of the following general form:

$$H_{ATi} - H_{BTj} = +\frac{a}{g}(V_{ATi} - V_{BTj}) \quad \ldots\ldots\ldots\ldots \text{(7a)}$$

$$H_{BTi} - H_{ATj} = -\frac{a}{g}(V_{BTi} - V_{ATj}) \quad \ldots\ldots\ldots\ldots \text{(7b)}$$

The notation appears complicated, but it seems that this is the only way in which all the information can be given. The term H is the total pressure and V is the velocity. The letters A and B refer to two points on a conduit, say a distance x feet apart. The term T_i indicates a time after the valve begins to close and T_j a time x/a seconds later. The time, x/a seconds, is that required for the pressure wave to travel from A to B. Thus, these equations relate the pressures and velocities at two points in the same conduit, the pressure and velocity at one point being x/a seconds earlier or later than at the other point, where x is the distance between the points and a is the wave velocity.

It is convenient many times to use pressure and velocity ratios

instead of the actual velocities. This can be done by dividing the left and right side of Equations 7a and 7b by H_0 and multiplying the right side by $\frac{2V_0}{2V_0}$. These equations then become:

$$h_{ATi} - h_{BTj} = 2\rho(v_{ATi} - v_{BTj}) \quad \text{.....(8a)}$$

$$h_{BTi} - h_{ATj} = -2\rho(v_{BTi} - v_{ATj}) \quad \text{.....(8b)}$$

where $h = \frac{H_T}{H_0}$ $v = \frac{V_T}{V_0}$, and $\rho = \frac{aV_0}{2gH_0}$ which is known as the pipe

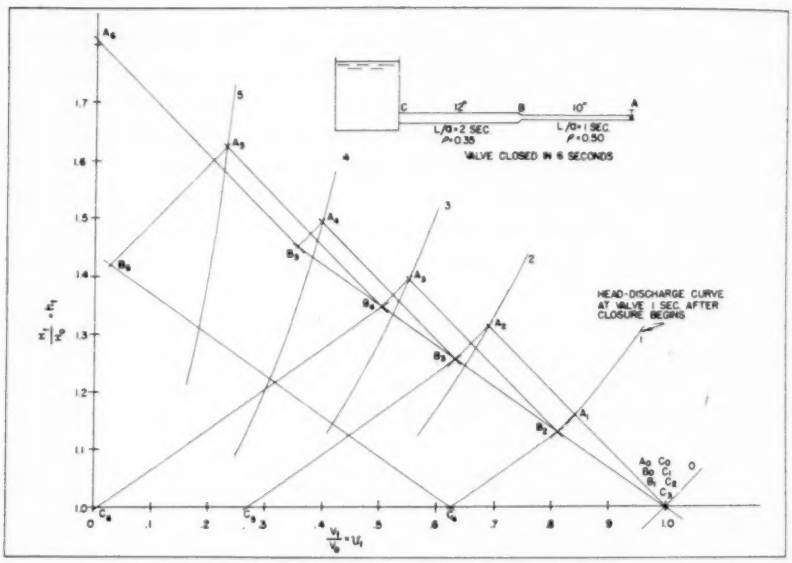

FIG. 5. Graphical Solution for Water-Hammer Pressure in a Compound Pipe for Gradual Gate Closure; friction neglected

line characteristic. Equations 7 or 8 and the equation giving the relation between head and velocity, $V_t = B_t\sqrt{H_t}$, at the gate, when solved simultaneously, permit the calculation of the water-hammer pressure at any point at any time in a simple, compound or branched conduit. Of course, these equations can be solved analytically or graphically; however, the graphical method is simpler and in general takes less time.

A simple water-hammer problem in a two diameter compound pipe will be solved graphically to illustrate the general principles. The sketch of the conduit and the graphical construction are shown in

fig. 5. The equations of the type 8a and 8b between points A and B, and between points B and C, together with the equation giving the relation between head and discharge at the gate are the equations to be solved simultaneously at points A and B. Of course, the pressure at point C remains at H_0. Note that in this case the relative head and velocity are used instead of the actual values. However, this does not alter things in any way. Since the effects of valve closure are not felt at point C until 3 seconds after closure begins, point C_3 is located on the point representing initial conditions. Note that the maximum pressure rise at B is considerably below that at A, which is as expected. In this case, friction was not considered. The friction loss can be taken into account quite readily in this graphical solution, by considering it as concentrated at various points. In this particular problem, it would be best to consider the loss in pipe CB as concentrated at C, and the loss in pipe AB as concentrated at B. For initial conditions then, B would be below A a distance equal to the loss in pipe AB, and C would be below B a distance equal to the loss in BC. The C points would all lie on the curve below the H axis a distance equal to the total loss in pipe AC. The maximum pressure at A and B would be reduced of course, depending on the relative value of the friction loss compared to the static head.

Pump Discharge Lines

The study of water hammer in centrifugal pump discharge lines is a most important problem in the field of water works pipe design. Pump discharge lines are always provided with a gate, cone, needle or check valve near the pump; and unless such valves are properly adjusted and operated, the pressure rise in the water line may be very great. The most important water-hammer problem relating to centrifugal pump discharge lines is that created when the power to the pump is suddenly shut off for some reason or fails unexpectedly.

Schnyder (5) divides the phenomenon which takes place during surge conditions in pump discharge lines into three phases as follows: First, right after power failure the speed of the pump decreases and also the discharge. As the pressure drops, the water slows down and begins to reverse in direction. The second phase is the period from the start of reversal of flow until maximum discharge is reached in the reverse direction. The third and final phase is the period between maximum backward flow and complete stopping of this backward flow.

During the first phase, the sudden drop in pressure can cause a break in the water column in the upper part of the pipe line or at a point where the pressure approaches absolute zero. Such low pressures may, of course, damage the conduit, therefore, it is necessary to limit the rate of change of the water velocity by installing surge tanks or air inlet valves at the upper parts of the pipe line. The use of a heavy flywheel effect on the pump and motor will do much to decrease the rate of slowing down of the water column. If the discharge valve is closed before the water column comes to rest in the first phase, the drop in pressure will be increased.

During the second phase, various things can happen depending on what control and relief devices are installed at the pump. If only an ordinary swing-check valve is used, no reversal of flow is possible. The pressure then drops till the velocity becomes zero, after which the pressure will surge up. In general, the pressure rise under these conditions will be equal to the pressure drop below normal. In such a case, the pressure may reach twice the normal head, if there has been no breaking of water column. However, if the water column breaks due to the pressure reaching absolute zero, then, when the reverse flow occurs, the pressure may rise to a much higher value than twice the normal head. If a surge suppressor is used in combination with the check valve, a different condition results. The surge suppressor opens when the water column starts to reverse and the check valve closes. The pressure rise is controlled by closing the surge suppressor slowly. The surge-suppressor might start to open during the first phase in order to limit the drop in pressure.

Instead of a check valve and surge-suppressor, a controlled closure valve might be used. This valve would be closed rapidly during the first phase, and then more slowly starting at the beginning of the second phase. The rate of closure should be so predetermined as to limit the pressure rise to the maximum allowable. With a quick-closing check valve, there is no third phase, which is the period when the water that is flowing back towards the pump is being stopped.

A surge-suppressor may be of the air-chamber type or may be a spring loaded valve which opens up and discharges water. In either case, the suppressor acts in such a manner as to stop the back flow gradually and thus limit the pressure rise. It should be remembered that to avoid excessive surges, the backward flow during the second phase cannot be restricted too much, nor shut off too quickly during the third phase.

An important item influencing the speed with which the water column in the discharge line will reverse is the inertia of the pump and motor. With a pump of small inertia, the velocity drops very rapidly, and reversal will occur in a time which may be less than $2L/a$ seconds. With a pump in which there is present considerable inertia of the rotating parts, reversal will not occur for a number of $2L/a$ intervals.

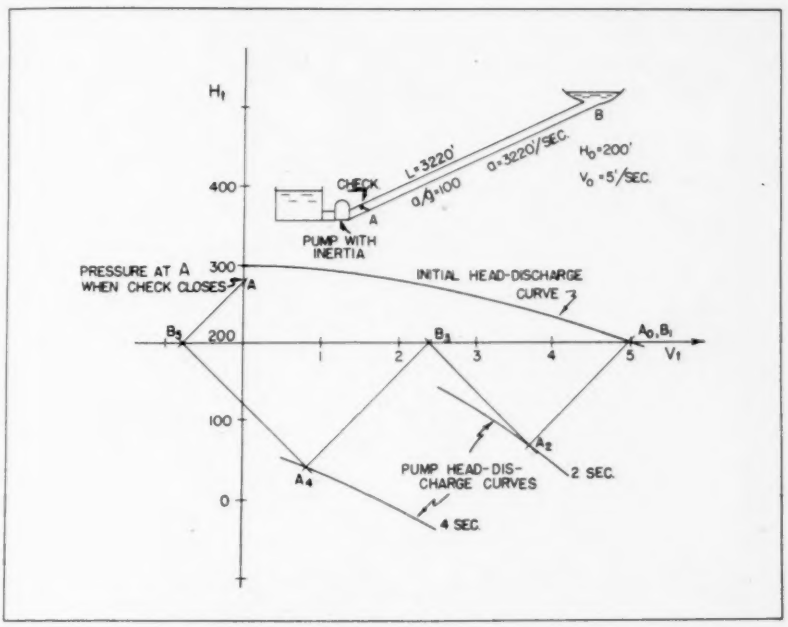

FIG. 6. Graphical Solution for Water-Hammer Pressure in Centrifugal Pump Discharge Line Equipped with Check Valve after Power Fails to Pump

To illustrate the method of graphical solution of water-hammer problems in pump-discharge lines, two problems will be solved: One where the line is equipped only with an ordinary check, and the other where a controlled valve closes according to a definite law. The first case is shown in fig. 6. Equation 7 is the basis for the graphical construction. The pump has some inertia and it takes over two $2L/a$ intervals to decrease the velocity to zero. Of course, at the check valve, the velocity is zero toward the pump since it is assumed that the check valve closes at the instant of reversal. A check valve that closes when reversal begins will not ordinarily cause

a pressure rise above twice the normal operating head, H_0, even for a pump without much inertia.

In fig. 7 is shown the graphical construction for the condition when an automatic valve closes gradually when the power fails to the pump. Of course, to solve this problem, the head-discharge relationships for the valve must be known and also those for the pump, both for normal rotation and also for reverse rotation. The valve should be closed before a high velocity toward the pump is developed, otherwise high pressures will result when the valve is finally

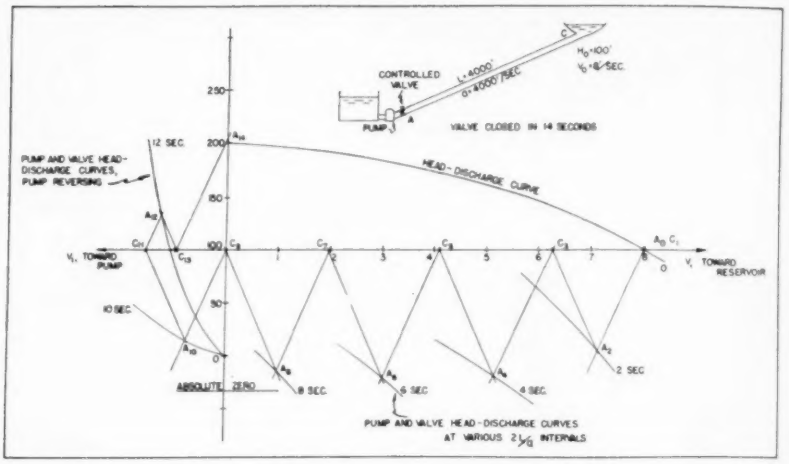

FIG. 7. Graphical Solution for Water-Hammer Pressure in Centrifugal Pump Discharge Line Equipped with a Controlled Cone Valve after Power Fails to Pump

closed. Such controlled valves may produce higher water-hammer pressures than check valves if they are not operated correctly.

Branched Pipes

Many piping arrangements with which the water works system designer is concerned are far from being simple conduits. The complete solution of water-hammer problems in piping systems having various sizes of pipes together with numerous branches becomes almost impossible. The graphical method of solution can be used for branching conduits; however, the construction becomes extremely complex and difficult to follow. The authors believe that in certain of these problems an analytical method, presented by Thomas

in a discussion of a paper by Angus (4), is more easily followed and can be used just as conveniently and as accurately.

This analytical calculation is based on the solution of the same simultaneous equation on which the graphical method of calculation depends. Its use is quite simple once the reader becomes familar with the nomenclature and the arrangement of the computations.

Note that Equation 7b can be written thus:

$$H_{ATn} + SQ_{Atn} = H_{Btm} + SQ_{Btm} = C \quad \ldots\ldots\ldots\ldots (9)$$

$S = \dfrac{a}{gA}$, where A is the area of the pipe

Q = Discharge = VA

C = a constant

The letter A in the subscript designates one end of the portion of the conduit under consideration and B is the other end. The sign of S is plus if the positive direction of flow is toward A and negative if away from A.

Equation 9 is one of the general simultaneous equations to be used in the analytical solution, the other is the relation between head and discharge. Equation 6 can be written as:

$$Q = k\sqrt{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where k is a known function of the time of valve closure.

In order to demonstrate how the analytical solution is carried out, a problem shown in fig. 8 will be worked. There are three conduits in this problem, each having a different value of S, as shown. The two ends of the X conduit will be designated as A and B_A, that of the Y conduit as C and B_C, and that of the Z conduit as D and B_D. The pressures at B_A, B_C, and B_D will be assumed equal, changes in velocity head being neglected. Let $H_0 = 100$ feet, $Q = 10$ cu.ft. per sec., and total time of closure of valve at $A = 5$ seconds. The valve will be closed uniformly. Let the values of head and discharge be computed for one-second intervals. Friction loss is to be neglected.

For the X conduit at A we have the equations $H_A + S_X Q_A = C$ and $H_A = Q_A^2/k^2$, and therefore the discharge at any given time, t, is:

$$Q_A = \frac{Sk^2}{2}\left(\sqrt{1 + \frac{4C}{S^2k^2}} - 1\right) \quad \ldots\ldots\ldots\ldots (11)$$

The calculations for the X conduit are then listed in table 2. It should be noted that at first the calculations in this table can be made only for A_1 and A_2; it is then necessary to obtain the pressure and discharge at B_{A2}, B_{A3}, B_{A4}, by considering conditions at point B.

At the junction, Point B, we have that S_x is negative, while S_y and S_z are positive. Since the law of continuity holds, $Q_{BA} = Q_{BC} + Q_{BD}$. We also have $H_B + S_xQ_{BA} = C_x$; $H_B + S_yQ_{BC} =$

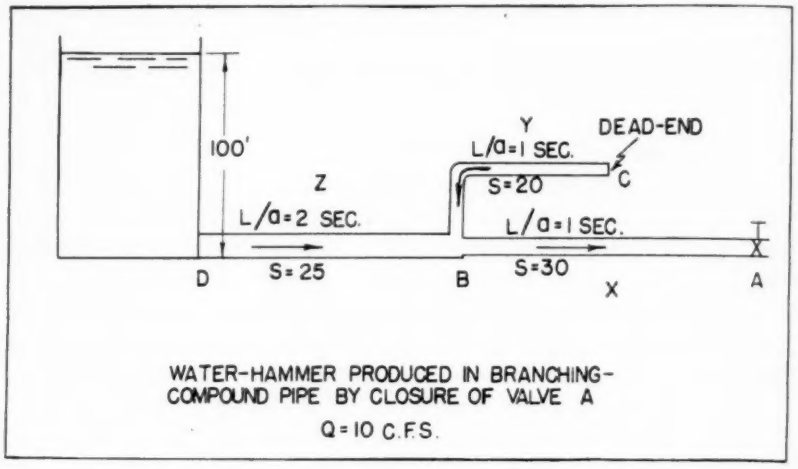

FIG. 8. Compound-Branching Pipe for Which Water-Hammer Pressure is Calculated by Analytical Methods

C_y; $H_B + S_zQ_{BD} = C_z$. The head at B then at any instant is given by:

$$H_B = \frac{\frac{C_x}{S_x} - \frac{C_y}{S_y} - \frac{C_z}{S_z}}{\frac{1}{S_x} - \frac{1}{S_y} - \frac{1}{S_z}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

(Remember that in this case S_x is negative)

The values of C_x, C_y, and C_z can be computed from the head and discharge at the other ends of the conduit at instants L/a sec. earlier. The discharges are obtained quite readily from equations between H_B, S and C listed above. The computations for point B, as tabulated in table 3 can only be carried up through the values at points B_A, B_C, and B_D, three seconds after valve closure starts. To ob-

tain the pressures at B_{A4}, B_{C4} and B_{D4}, conditions at point C must now be considered in order to calculate the pressure and discharge at C_3.

TABLE 2

Solution for Conduit X, Point A

ITEM NO.	POINT AND INSTANT, $T - \frac{L}{a}$	B_{A0}	B_{A1}	B_{A2}	B_{A3}	B_{A4}
1	Discharge, Q_B	10.0	10.0	8.55	6.75	4.8
2	Head, H_B	100.0	100.0	116.0	136.0	176.0
3	$C = H_B + 30Q_B$	400.0	400.0	372.8	338.5	320.0
4	k	0.8	0.6	0.4	0.2	00.0
5	Q_A (Eq. 11)	9.02	7.78	5.66	3.12	00.0
6	$H_A = C - 30Q_A$	131.0	166.0	202.8	245.0	320.0
	Point and Instant, T	A_1	A_2	A_3	A_4	A_5

TABLE 3

Solution for Point B

ITEM NO.	POINT AND INSTANT $\left(T-\frac{L}{a}\right)$	A_1	C_1	D_0	A_2	C_2	D_1	A_3	C_3	D_2
1	Discharge, Q_A, Q_C, Q_D	9.02	0	10	7.78	0	10	5.66	0	10
2	Head H_A, H_C, H_D	131	100	100	166	100	100	202.8	132	100
3	C_x, C_y, C_z	−141	100	350	−68	100	350	30.0	132	350
4	Head, h_B (Eq. 12)	116	116	116	136	136	136	176	176	176
5	Discharge, Q_{BA}, Q_{BC}, Q_{BD}	8.55	−.80	9.35	6.75	1.80	8.55	4.80	2.20	7.00
	Point and Instant, T	B_{A2}	B_{C2}	B_{D2}	B_{A3}	B_{C3}	B_{D3}	B_{A4}	B_{C4}	B_{D4}

TABLE 4

Solution for Point C

ITEM NO.	POINT AND INSTANT, $T - \frac{L}{a}$	B_{C2}	B_{C3}	B_{C4}
1	Discharge, Q_{BC}	−0.80	−1.80	−2.20
2	Head H_b	116	136	176
3	Head H_C	132	172	220
4	Discharge, Q_c	0	0	0
	Point and Instant, T	C_2	C_4	C_5

Considering point C, S_y is negative; Q_c is zero; and $H_c = H_{BC} - 20Q_{BC}$ where the H and Q at point B_C are one second earlier than at point C. The calculations are tabulated in table 4.

To complete the solution the relations in conduit Z at point D must finally be used. At this point, $H_D = 100$, $S = -25$, and $C = H_{BD} - 25Q_{BD}$, the values of Q_D and H_D are computed from $C = H_D - 25Q_D$ where H_D and Q_D represent conditions at D two seconds later than at B_D. The tabulations for the solution are shown in table 5.

The computations are started by writing into the tables all the known initial values of H and Q. The computations are then carried forward by filling into the tables computed values of H and Q from other tables. Once the calculations are started by making out the tables and by putting in the proper points whose head and discharge are related, the rest of the computation is relatively simple. The main advantage of working water-hammer problems by this

TABLE 5

Solution for Point D

ITEM NO.	POINT AND INSTANT, $T - \frac{L}{a}$	B_{D2}	B_{D2}	B_{D2}
1	Discharge, Q_{BD}	9.35	8.55	7.00
2	Head, H_{BD}	116.0	136.0	176.0
3	$C = H_{BD} - 25Q_{BD}$	−118.0	−78.0	1.0
4	Q_D	8.72	7.11	3.96
5	H_D	100.0	100.0	100.0
	Point at Instant, T	D_4	D_5	D_6

method over the graphical method is that greater accuracy is possible, and also someone else may easily check the computations.

Theoretically, problems of any complexity can be solved by this or the graphical method. However, as the number of transition sections, dead-ends, junctions, valves, etc., increases, the number of equations which must be solved simultaneously becomes large, thus making the computations very tedious.

In the analytical solution, the total friction in a pipe must be considered as concentrated at some point or points, as was done in the graphical method. Another table must be made out then for the "friction point" similar to the tables made out for points such as junctions, dead-ends, etc.

Relief Valves

If a relief valve is installed at the end of a conduit near the gate and if the gate is suddenly closed, the water in the pipe is not

stopped, since the relief valve opens and allows the water to keep flowing till the pressure is sufficiently reduced for it to close.

The equation for discharge from a relief valve can be written as:

$$Q_R = C_R\sqrt{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

In general, C_R depends on the pressure H, since the opening in the relief valve may vary for different pressures. For a sudden (in time less than $2L/a$) gate closure, the rise in water-hammer pressure is given by $\frac{a}{g}(V_0 - V)$, where V_0 is the initial velocity and V the velocity in the pipe due to the discharge through the relief valve. By combining Equation 13 with the expression for pressure rise due to instantaneous velocity destruction, it is possible to solve for H, the total maximum pressure, in terms of the area of pipe, A, the static pressure, H_0, and the value of C_R which, of course, depends on the type and size of relief valve and on the pressure H. Since $H - H_0 = \frac{a}{g}\left(V_0 - \frac{Q_R}{A}\right)$, by substitution from Equation 13 for Q_R we obtain after some manipulation:

$$H = \frac{x^2}{2} - \frac{x}{2}\left(\sqrt{x^2 + 4y}\right) + y \quad \ldots\ldots\ldots\ldots\ldots (14)$$

$$x = \frac{aC_R}{gA}, \text{ and } y = \left(H_0 + \frac{aV_0}{g}\right)$$

The solution may be a trial and error one since C_R ordinarily will vary with H. This equation was first proposed by Enger (6) and has been found to agree with experiments. However, remember that this equation applies only for a sudden valve closure and with the relief valve located at the gate. Friction has been neglected; its presence will, of course, decrease the maximum pressure developed.

If the gate closure is gradual, then the solution for maximum pressure rise must be made according to the graphical or analytical method previously discussed. To illustrate the calculation of such a problem, a graphical solution is made in fig. 9, assuming that a gate closes linearly in 3 seconds and the value of $2L/a$ is 1.5 seconds. The other data for the problem are: $H_0 = 200$, $V_0 = 7$ feet per second, $\frac{a}{g} = 100$. When the pressure H_t, reaches the value 300 ft., the relief valve opens. The discharge in the pipe is then made up of the discharge through the gate and that through the relief valve.

The general equations to be used in the solution are Equations 3 and 7. The head-discharge curves are plotted so that up to 300 ft., the discharge is just that through the gate, and above that it is through the gate and the relief valve.

Air Chambers

Water-hammer problems involving air chambers may be solved a number of ways depending on the conditions of the problem. The most common type of problem is that where a valve closes instantly,

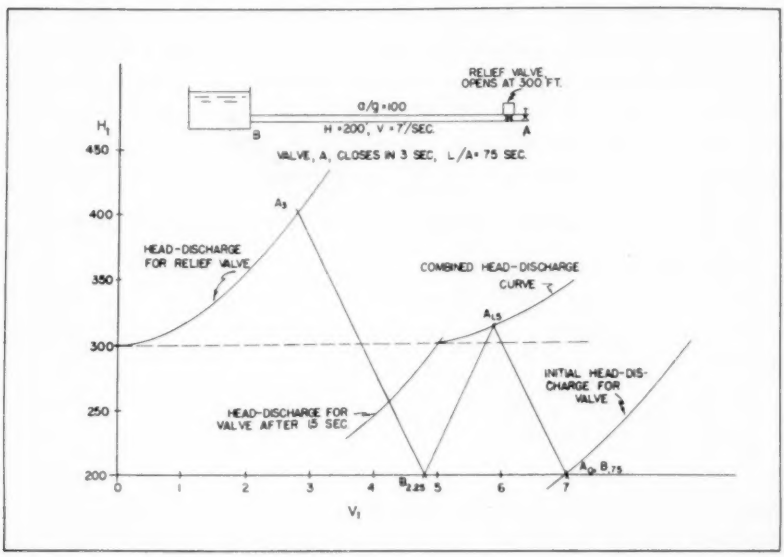

FIG. 9. Graphical Solution for Water-Hammer Pressure in Simple Conduit Equipped with Relief Valve for Gradual Gate Closure

or within a time of $2L/a$, and the maximum pressure developed is required. The air chamber is to be located near the gate, since otherwise the solution of the problem becomes complicated, and also the chamber is not very effective unless it is near the gate. Angus (7) indicates the graphical solution of such a problem; however, the construction is quite involved. Allievi (8) develops an analytical type of solution which is quite convenient to use.

Unless the pipe line is extremely long, the authors believe that the best method of solving the air-chamber problem is by the arithmetic

step by step solution of the simultaneous equations controlling the phenomena. This method of solution is not exact for very long pipe lines, since it is based on the assumption that the velocity varies linearly in the interval $2L/a$; this is not correct for long intervals. The solution proposed is as follows:

The general Equation 5 relating the velocity and water-hammer pressure is one of the equations to be used, and the other is the equation relating pressure and volume in the air-chamber. If isothermal compression is assumed, the pressure-volume equation is:

$$(H_n + H_a)U_n = C \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

U_n = volume of air in cubic feet.

H_a = atmospheric pressure, feet of water.

The value of C is determined when $H_n = H_0$, and $U_n = U_0$.

Equation 5 is written thus, for h_n, the water-hammer pressure rise:

$$h_n = (H_n - H_0) = \frac{a}{g}(V_{n-1} - V_n) - (H_{n-1} - H_0) \ldots (15a)$$

Recall that the subscript n refers to various time intervals of $2L/a$ seconds. The value of U_n during any interval is best determined thus:

$$U_n = U_{n-1} - (V_{n-1} + V_n)\frac{A2L}{a} \quad \ldots\ldots\ldots\ldots (16)$$

A = area of pipe in sq.ft.

Equation 16 states that the air volume at any interval is equal to the volume at the previous interval minus the amount of water that flowed into the chamber during the interval, $2L/a$ seconds.

Equations 15, 15a, and 16 are the simultaneous equations that must be solved. By starting with the first interval and finding H_1, V_1, and U_1, and then finding H_2, V_2, U_2, etc., the solution becomes quite simple, especially if the results are tabulated conveniently.

In table 6, are shown the values of U, V, and H, for a problem having the following data: static head, $H_0 = 50$; $H_A = 32$ ft.; $U_0 = \frac{1}{4}$ cu.ft.; $L = 300$; $a = 4{,}000$; pipe diameter = 2 in.; $V_0 =$ 10 ft. per second. The best method of solution is by trial and error and procedure is as follows: Assume a value of U_1, for instance, then solve Equation 15 for H_1. Then solve Equation 15a for V_1, and check the assumed value of U_1 by using Equation 16.

Note that the maximum pressure developed is about 235 feet and occurs between the 7th and 8th intervals. The maximum water-hammer pressure possible in this pipe line without any air-chamber for instantaneous valve closure is $\left(\frac{a}{g} \times 10\right) = 1{,}240$ feet, or a total pressure of 1,290 feet.

If pipe friction is to be taken into account in Equation 15a, instead of the static head, H_0, the quantity $(H_0 - KQ^2)$ would be used where K is a constant as determined from the pipe friction formula. This complicates the solution; however, friction should be taken into account if the initial flow head is only a small part of the static head.

TABLE 6

Solution of Air-Chamber Problem

n	U_n	H_n	V_n
0	0.25	50.0	10.0
1	0.2175	62.30	9.901
2	0.1857	78.39	9.573
3	0.1554	99.92	8.943
4	0.1278	128.41	7.910
5	0.1045	164.17	6.360
6	0.0872	203.09	4.209
7	0.0778	231.50	1.516
8	0.777	231.87	−1.409
9	0.0867	204.45	−5.525

If the gate is closed gradually instead of instantaneously, the calculation becomes quite involved. Another simultaneous equation must be satisfied then, that giving the relation between the total pressure and the discharge through the gate at any instant.

In order to determine the necessary size of air-chamber to keep the pressure rise below a certain maximum, the above method of analysis is impractical to use, since various sizes of air-chambers would have to be assumed and the maximum pressure rise for each one calculated. This would be a much too tedious procedure. For this reason the air-chamber problem is analysed from the point of view of energy considerations. Such an analysis is not quite exact since the entire moving water column is considered to move at the same velocity at any instant. This, of course, is not true. Due to

the compressibility of the water and the elasticity of the pipe, water velocity at one point is quite different from that at another point at any instant.

The energy analysis was first presented by Enger (6), and shows reasonable agreement with experimental data. The analysis in general is as follows: The energy stored in the air-chamber will be approximately equal to: (A) the kinetic energy of the moving column of water, E_k; (B) plus the potential energy of the water entering the air-chamber, E_p; (C) minus the energy loss that the water entering the air-chamber undergoes due to pipe friction, E_f; (D) minus the potential energy stored in the pipe walls and in the water, ΔE. The above terms are equal to the following expressions:

$$E_k = \frac{LAw}{2g} V_0^2 \quad \text{(17a)}$$

$$E_p = wH_s'(U_0 - U_m) \quad \text{(17b)}$$

$$E_f = cw(U_0 - U_m)KV_0^2 \quad \text{(17c)}$$

The term H_s' is the absolute static pressure in feet; U_0 and U_m are the air volumes under initial conditions and under conditions of maximum pressure respectively; KV_0^2 is the initial friction loss and c is some factor less than unity. The reason for use of the coefficient c is that the friction loss varies with time as the velocity decreases, and each particle of water entering the chamber undergoes a different loss. However, since the rate of water entrance into the air chamber is high when the friction is large, that is, when the velocity is high, this coefficient has a value under ordinary conditions of about 0.70 to 0.90.

The total energy stored in the chamber should be equal to the work done. For isothermal compression, the work done is equal to:

$$W = wH_0' U_0 \log_e \frac{U_0}{U_m} \quad \text{(18)}$$

H_0' = absolute initial pressure in feet

Of course, the compression may not be isothermal; however, it is simpler to consider it this way, especially since the whole analysis is of an approximate nature. The following equality is then set up:

$$W = \frac{LAwV_0^2}{2g} + wH_s'(U_0 - U_m) - cw(U_0 - U_m)KV_0^2 - \Delta E \quad \text{(19)}$$

To simplify the analysis, c will be considered as unity and ΔE will be dropped. These errors tend to balance each other. Also, ΔE is small if the air-chamber produces any large reduction in pressure. Since $KV_0^2 = (H_s' - H_0')$, where H_0' is the initial absolute flow pressure, we have:

$$wH_0' U_0 \log_e \frac{U_0}{U_m} = LAwV_0^2 + H_0' w(U_0 - U_m) \quad \ldots\ldots (20)$$

Solving for U_0, there is obtained:

$$U_0 = \frac{LAV_0^2}{2gH_0'\left(\text{Log}_e N + \frac{1}{N} - 1\right)} \quad \ldots\ldots (21)$$

$$N = \frac{H_m'}{H_0'}$$

H_m' = Total Maximum pressure, absolute.

The above equation is dimensionally correct, and can be written thus:

$$\alpha = \frac{1}{\beta} \text{ where,}$$

$$\alpha = \frac{2gH_0' U_0}{LAV_0^2} \quad \ldots\ldots (22)$$

$$\beta = 2.3 \log N + \frac{1}{N} - 1 \quad \ldots\ldots (23)$$

Using the above equation between α and β, the value for the maximum pressure for the illustrative problem previously solved analytically in table 6 is 238 ft. The value obtained analytically was 235 ft. Note that the terms α and β are dimensionless parameters. In fig. 10 is shown a plotting of α and a β as computed from experimental data obtained by Kessler (9), Enger (6), and the authors. Kessler's data were obtained on 1-inch galvanized iron pipe which varied in length from about 50 feet to 400 feet. Enger's data were obtained on 2-inch pipe, 730 feet long. The authors' data were obtained on 1-inch iron pipe, about 100 feet in length.

The straight line in fig. 10 has a slope of -1, indicating the relation $\alpha = \frac{1}{\beta}$. The data which curve off this straight line were ob-

tained by Kessler for small air-chambers using high velocities of flow, thus resulting in large values of N. As was noted in the above analysis, if the pressure reduction is relatively small, a considerable amount of the energy is stored in the pipe and in the water, resulting in a large value for the (ΔE) term which we neglected. Nevertheless, it appears that a plotting of α and β correlate the data quite well. For values of β from .01 to 1, or values of N from slightly above 1 to about 6, and for values of α from 1 to 100, the relation $\alpha = \frac{1}{\beta}$ can be used. Figure 10 can be used for determining air-chamber sizes for a wide range of conditions with a fair degree of certainty as to the results to be obtained.

The data in fig. 10 are for conditions when the opening into the air-chamber is of the same size as the pipe, and there is very little loss of energy between the pipe and the chamber. It is possible to make an air-chamber more effective by the introduction of a restriction, or any other energy dissipating device, between the pipe and the chamber. However, there is a limit to how much friction can be introduced between the pipe and chamber without causing an increase in the pressure instead of a decrease. There is undoubtedly some optimum value for any given set of conditions. However, this problem needs more investigation before any predictions can be made as to the possible effects of this introduction of friction.

The plotting of α and β does not, of course, take into account all the variables that may affect the water-hammer pressure developed. For instance, variation in pipe wall thickness and material is not taken into account, primarily on the assumption that the effects introduced by such variations will be small if the air-chamber produces an appreciable pressure reduction. Figure 10 is an attempt to present available data conveniently for practical use without resorting to extensive tables or complicated equations.

Summary

The material presented in this paper may be summarized as follows:

1. The general theory of water-hammer pressure for instantaneous and gradual gate closure for a simple conduit is presented, and a simple example is calculated by the so-called "arithmetic integration" method.

2. The elementary principles of the method of graphical solution

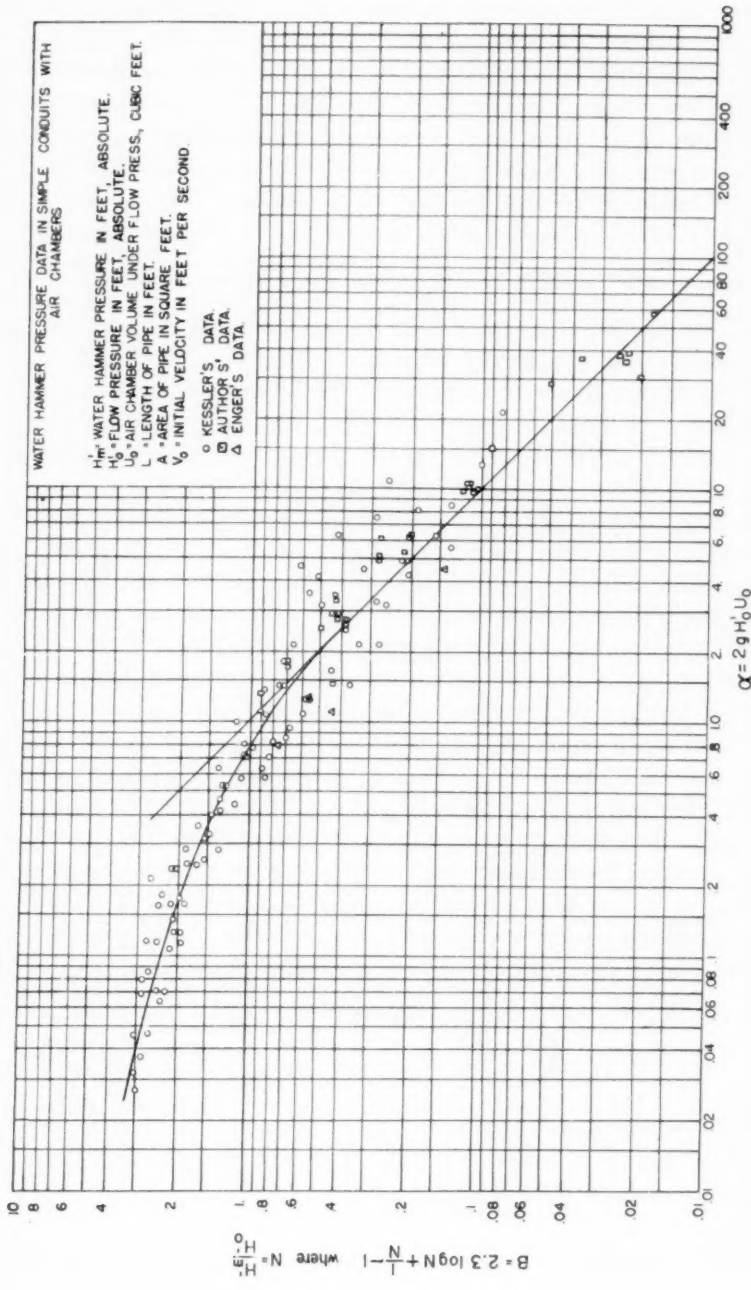
WATER HAMMER PRESSURE DATA IN SIMPLE CONDUITS WITH AIR CHAMBERS
H'_m = WATER HAMMER PRESSURE IN FEET, ABSOLUTE.
H'_0 = FLOW PRESSURE IN FEET, ABSOLUTE.
U_0 = AIR CHAMBER VOLUME UNDER FLOW PRESS., CUBIC FEET.
L = LENGTH OF PIPE IN FEET.
A = AREA OF PIPE IN SQUARE FEET.
V_0 = INITIAL VELOCITY IN FEET PER SECOND.
○ KESSLER'S DATA.
□ AUTHOR'S DATA.
△ ENGER'S DATA.
$\alpha = \frac{2gH'_0U_0}{LAV_0^2}$
$B = 2.3 \log N + \frac{1}{N} - 1$ where $N = \frac{H'_m}{H'_0}$

FIG. 10. Plotting of Experimental Data Relating to Water Hammer in Simple Conduits Equipped with Air Chambers

for water-hammer pressures are explained, the simultaneous equations controlling the related phenomena are presented, and a simple problem is worked. The effect of friction on the development of water-hammer pressure is indicated.

3. The solution of water-hammer problems in compound pipes is next taken up. A slightly different type of graphical solution is employed; the details of which are thoroughly discussed, and a simple problem is solved.

4. The phenomena occurring in centrifugal pump discharge lines are analysed and the effects of check valves, surge-suppressors, and controlled valves, are discussed. Two illustrative problems are worked out, one where the discharge line is equipped only with a swing check, and the other where there is a controlled valve closing according to a definite law, when the power to the pump fails.

5. The complicated problem of water-hammer in simple branching conduits is presented. An arithmetical, instead of graphical method of solution is suggested as more desirable for the solution of such problems. An illustrative example of a compound pipe having a dead-end branch is worked out to indicate the method of this arithmetic solution.

6. The theory of mechanical relief valve operation is next discussed. The equation relating relief valve opening, conduit size, initial pressure and velocity, and the final pressure developed in the pipe for instantaneous gate closure is derived. A problem involving gradual gate closure where the relief valve opens at a definite pressure is worked out graphically.

7. The air-chamber problem is the last part of the paper. An illustrative example is given of the step-by-step method of the arithmetic solution for determining the maximum water-hammer pressure in a line equipped with an air chamber and when instantaneous valve closure occurs. The energy equation for solving the air-chamber problem is derived, and experimental data are presented to check this equation. A simplified method of calculating the necessary air-chamber volume is indicated by use of a dimensionless plotting of the experimental data.

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Pressure Air Chambers in Centrifugal Pumping

By W. L. Boerendans

IN THE following paper, the use of an air chamber to reduce water-hammer pressure caused in a pipe line by a sudden shutdown of a centrifugal pump will be considered.

The air chamber may also serve another purpose, that of equalizing irregularities in the distribution system (1). We must emphasize the fact that, even though for a special case we conclude that the use of an air chamber does not seem a reasonable method of reducing water-hammer pressure, this does not preclude its desirability for some other purpose. The opinions as to the value of air chambers as a pressure reducing device vary considerably. Lorenzo Allievi, the formulator of the pressure oscillation theory, states in his book, "In order to reduce the magnitude of the water shocks, air chambers are often placed near the regulating valve, above the pipe, with the assumption that a part of the energy of the water column will be absorbed by the compression of the air caused by this pressure rise."

Practical experience has shown that the air chamber does not entirely satisfy the functions ascribed to it.

In order to give a more recent opinion we refer to a lecture by Oberbaurat O. Hanneman (2), "In the pipe line, in order to reduce the pressure oscillations which occur when the switch is suddenly closed, check valves are installed, the first of which is immediately outside the plant. In the same system, the air chamber is removed, as it increased the variations caused by closing the switch."

A paper by W. L. Boerendans, Engineer, Amsterdam, which was published in the magazine "Water" (Neth.) in the issues of Jan. 27, Feb. 10 and Feb. 24, 1939. Translation is by: E. A. Menuez, Research Fellow, Dept. Hydraulic and Sanitary Engineering, University of Wisconsin; assisted by Miss Jennie Reitsma of The Haag, Netherlands, Graduate Student, University of Wisconsin; under the direction of L. H. Kessler, Associate Professor of Hydraulic and Sanitary Engineering, University of Wisconsin. This translation is published by permission of the Editor of "Water," B. F. Van Nievelt, and the author.

To these quotations could be added many more, and for praise of the air-chamber method of reducing pressure shock see *Water* (Neth.), No. 17, (1934), also *Das Gas und Wasserfach* (Ger.), No. 19, (1933). From these articles it may be seen that the method of computation of the influence of the air chambers is important.

Although air chambers are frequently used, we can find little information in literature as to the method of computation of the desired size or their effect on the pressure variations.

In general one starts with the assumption that the water in the pipe and the pipe itself are inelastic. The water column which is in movement is considered as one compact mass, which at one side is under the influence of the pressure in an air chamber and on the other the pressure in a reservoir or standpipe. When the delivery pump is stopped, the movement of the water mass, which has not stopped, causes the pressure in the chamber to reach a minimum then increase again as the velocity of the water mass changes direction and the water, under the pressure of the reservoir, is again pressed into the chamber.

On this mass we can apply the law of motion—force equals mass times acceleration—and arrive at a differential equation which is only approximately solvable. In order to solve this differential equation, we apply an approximation which is rarely permissible.

This first equation is of the second degree. It can, without approximation, be transformed into an equation of the first degree which enables us to draw more general conclusions than from the first equation.

Both solutions depend upon the erroneous assumption that water and pipe are incompressible. On the computation of the pressure variations with an air chamber on the line Allievi writes "An analytical formulation of these occurrences is practically impossible because of the very complicated relationships. Only in the case of complete closure of the valve can an approximate theory of the hydrodynamic phenomenon be developed which takes care of the operation of the air chamber, the elasticity of the pipe material and compressibility of the fluid."

This method of treatment of the problem is justified by the observation that the oscillation period with the air chamber installed is significantly larger than the period, $4L/a$, obtained from consideration of elastic pipe and liquid with no air chamber. When we assume both pipe wall and liquid completely inelastic, from hydro-dynamical

theory, the period T of oscillation of the air cushion in the chamber, under action of the checked mass of water will be:

$$T = 2\pi \sqrt{\frac{LwV_0}{F_1 P_0 g}}$$

From this equation we can see that T is much greater than $4L/a$ because

$$\frac{V_0}{F_1 L} > \frac{4P_0 g}{\pi^2 a^2 w} \quad \text{and} \quad \frac{V_0}{F_1 L} > \frac{4}{10^9} P_0$$

One oscillation will develop of period T and superimposed on this will be secondary oscillations of the period $4L/a$. We have now a slow primary oscillation (due to the action of the air cushion in the chamber) and rapid secondary oscillations (developing out of the elasticity of fluid and pipe).

Later a graphical method is developed which enables the computation of the influence of air chambers without differential equations.

As this last method is not readily adapted to the drawing of general conclusions, we will first discuss the old computation methods and after that the graphical method, drawing conclusions about the desirability of the use of air chambers.

When a stand pipe or reservoir must be maintained full, then usually, when the desired level is reached, the pump is stopped by an electrical pressure switch and a check valve in the line prevents the water flowing out of the line when the pump has stopped, creating water-hammer shocks. The air chamber is now expected to keep those pressure waves within allowable limits.

In general the frictional resistance to the flow of water will cause a reduction in the pressure rise. In order to consider the influence of the air chamber alone, it is necessary to neglect the effect of friction. A second assumption is made that the air expands and contracts isothermally. Whether or not this is true depends upon the area of the exposed surface of the air chamber. If the surface is sufficiently large that the heat generated by compression can be dissipated, then this is true; if the surface is too small, the compression lies somewhere between isothermal and adiabatic.

In order for the first two methods to apply it is necessary that compression be isothermal. With the graphical method, however, the expansion may take place according to any desired rule providing the curve is plotted for the particular law chosen.

First Method

In the first method described here the water and pipe material are considered inelastic. This is an approximate solution and is taken in part from H. Berg's "Kolbenpumpen."

When an air chamber is placed on the pipeline near the pump but ahead of the check valve, as the motor is cut off the check valve will close, because the pressure ahead of it is greater than the pressure from the pump side.

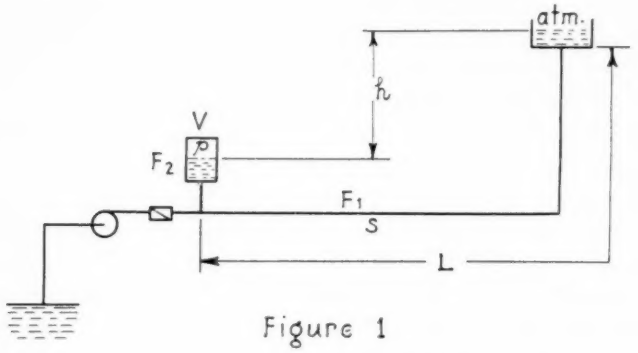

Figure 1

In order to arrive at a general equation which represents the movement of the water mass we assume the air chamber to be cylindrical with vertical center line and that:

F_2 = area of section of the air chamber,
F_1 = area of section of pipe,
L = length of pipe,
V = volume of air in air chamber,
V_0 = volume of air in air chamber at $t = 0$,
P = pressure of air in air chamber,
P_0 = pressure of air in air chamber for time $t = 0$,
w = unit weight of the liquid,
g = the acceleration due to gravity,
m = the mass of water in pipe,
x = the distance through which the center of mass of the liquid moves,
S = the velocity of the center of mass of the liquid,
S_0 = the velocity of the center of mass of the liquid at $t = 0$,
h = the head in air chamber at $t = 0$,
y = the vertical movement of the water surface in air chamber.

If the pump is operating and we neglect friction, the pressure in the air chamber will be:

$$P_0 = hw + A \text{ where } A \text{ is atmospheric pressure.}$$

Assuming isothermal expansion of air,

$$PV = P_0V_0 \text{ and } V = V_0 + F_2y \text{ and } F_2y = F_1x$$

The force of the water in the pipe is:

$$(P - P_0)F_1 \quad \text{hence} \quad (P - P_0)F_1 = m\frac{d^2x}{dt^2}$$

$$\text{or} \quad F_1(P - P_0) = \frac{LF_1w}{g}\frac{d^2x}{dt^2}$$

From which we may obtain the general differential equation:

$$\frac{Lw}{g}\left(\frac{d^2x}{dt^2}\right) + P_0 - \frac{P_0}{1 + \frac{F_1x}{V_0}} = 0 \ldots\ldots\ldots\ldots (1)$$

Because in the third term, x occurs in the denominator, this equation is insolvable for x.

We then must assume that the change in volume of air is small, then F_2y is small and F_1x is small. With this the case, we can replace

$$\frac{1}{1 + \frac{F_2y}{V_0}} \quad \text{by} \quad 1 - \frac{F_2y}{V_0}$$

and the equation becomes

$$\frac{Lw}{g}\frac{d^2x}{dt^2} + \frac{F_1x}{V_0} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

And the general solution of this equation is

$$x = C_1 \cos \sqrt{\frac{F_1P_0g}{LwV_0}}\,t + C_2 \sin \sqrt{\frac{F_1P_0g}{LwV_0}}\,t$$

in which C_1 and C_2 are arbitrary constants which may be evaluated by consideration of the initial conditions.

At time $t = 0$, $x = 0$ and $\frac{dx}{dt} = S_0$ by definition. Hence $C_1 = 0$ and as $\frac{dx}{dt} = C_2\sqrt{\frac{F_1P_0g}{LwV_0}} \cos \sqrt{\frac{F_1P_0g}{LwV_0}}\,t$

we have for $t = 0$,

$$C_2 = S_0 \sqrt{\frac{LwV_0}{F_1 P_0 g}} \quad \text{and} \quad \frac{dx}{dt} = S_0 \cos \sqrt{\frac{F_1 P_0 g}{LwV_0}}\, t$$

But

$$(P - P_0) = \frac{m}{F_1} \frac{d^2x}{dt^2}$$

Then

$$P - P_0 = -\frac{m}{F_1} S_0 \sqrt{\frac{F_1 P_0 g}{LwV_0}} \sin \sqrt{\frac{F_1 P_0 g}{LwV_0}}\, t$$

or

$$P - P_0 = -S_0 \sqrt{\frac{F_1 LwP_0}{gV_0}} \sin \sqrt{\frac{F_1 P_0 g}{LwV_0}}\, t \quad \ldots\ldots\ldots (3)$$

The pressure variations then may be represented by a sine curve of which the period is: $T = 2\pi \sqrt{\frac{LwV_0}{F_1 P_0 g}}$ and the minimum pressure will be: $P_{\min} = P_0 - S_0 \sqrt{\frac{F_1 LwP_0}{gV_0}}$

and

$$P_{\max} = P_0 + S_0 \sqrt{\frac{F_1 LwP_0}{gV_0}}$$

The conclusions which we can draw from these relations are: The pressure variations are greater as:

1. The initial velocity, S_0, is increased;
2. The initial head on the air chamber is greater;
3. The volume of fluid in the pipe is greater;
4. The air volume is smaller.

Validity of Approximation

The change of Equation 1 to 2 puts the work done on the moving column of water in stopping its motion directly proportional to the distance, x, of travel of the center of mass of the moving water, or to y, the change in level of the water in the air chamber.

This change is shown graphically in fig. 2. According to the law of isothermal expansion the change, y, of the water level would be according to the curve PQ. The approximation assumes that this ordinate changes according to the straight line RS which is

tangent to PQ at P_0. Consider at some point A the ordinate y, which should have the value AB, has under the approximation a value AC. The error is then BC. For a slight change in water level the error BC will be small, also in the case of a large volume of air, V_0, the curve PQ will tend to straighten and the error will also be reduced.

For an example assume a centrifugal pump connected to a line 5 km. long supplying a standpipe the water level in which is 40 m. higher than that of the pump. The air content of the chamber is assumed 1.2 cu.m. at $t = 0$. The pipe is 300 mm. in diameter and the velocity of water is 1 meter per sec.

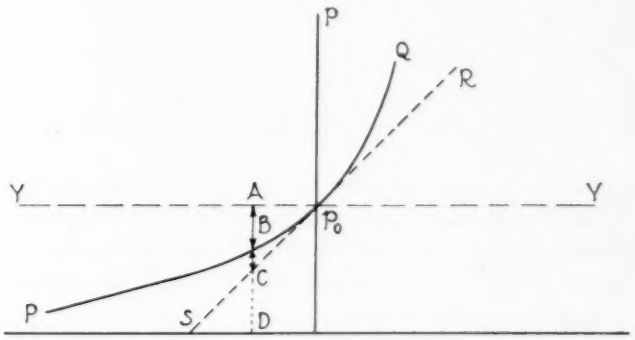

Figure 2

The static pressure in the air chamber is then 40,000 + 10,000 = 50,000 kg. per sq. meter.

Figure 3 gives the pressure curve according to Formula 3. The minimum pressure is

$$P_{\min} = 50{,}000 - 1\sqrt{\frac{0.071 \times 5000 \times 1000 \times 50{,}000}{9.81 \times 1.2}}$$

$$= 50{,}000 - 38{,}500 = 11{,}500 \text{ kg. per sq. meter or } 1.5 \text{ meters above atmospheric.}$$

and

$$P_{\max} = 50{,}000 + 38{,}500 = 88{,}500 \text{ kg. per sq. meter, or } 78.5 \text{ meters head.}$$

The period is $T = 2\pi\sqrt{\dfrac{5000 \times 1000 \times 1.2}{0.071 \times 50{,}000 \times 9.8}} = 83$ seconds

Second Method

The *second method* is without approximation, and both water and pipe are assumed inelastic.

The insolvable second degree differential equation can be reduced to the following first degree differential:

$$\frac{dx}{dt} = \left[\frac{2gP_0V_0}{LwF_1}\,\mathrm{Log_e}\left(1 + \frac{F_1x}{V_0}\right) - \frac{2gP_0}{Lw} + C_3\right]^{\frac{1}{2}}$$

in which C_3 is again qualified by the boundary condition, when $t = 0, \dfrac{dx}{dt} = S_0$

On solving we obtain:

$$S^2 - S_0^2 = \frac{2gP_0V_0}{LwF_1}\left(\mathrm{Log_e}\frac{P_0}{P} - \frac{P_0}{P} + 1\right) \quad \ldots\ldots\ldots (4)$$

which gives the relation between the velocity of the center of mass of the water in motion and the pressure in the air chamber.

The pressure in the air chamber will take its lowest minimum value when $S = 0$. These lowest values, which are the most important are given by the relation

$$\left(\mathrm{Log_e}\frac{P}{P_0} + \frac{P_0}{P} - 1\right) = \frac{LwF_1}{2gP_0V_0}S_0^2 \quad \ldots\ldots\ldots\ldots (5)$$

In fig. 4 is shown the relation between P/P_0 and the term on the left of Equation 5. From this we see that as $\left(\mathrm{Log_e}\dfrac{P}{P_0} + \dfrac{P_0}{P} - 1\right)$ increases all the pressure differences increase.

The conclusions which may be drawn from Equation 5 are:
The pressure variation is greater as:

1. The initial velocity is greater;
2. The pressure at the tower is less;
3. The contents of the pipe are greater;
4. The volume of air is less.

Conclusion No. 2 of this second method agrees with the second conclusion of the approximate method. In both methods it appears that there must be a definite relation between the volume of air and the content of the pipe at the moment of stopping.

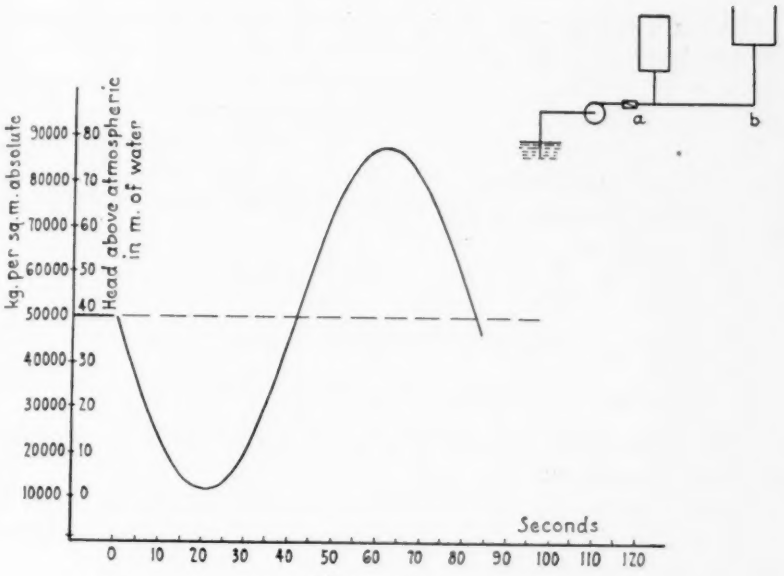

Figure 3

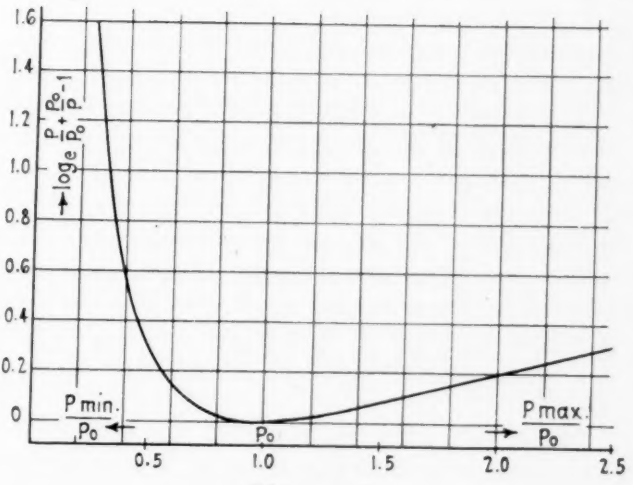

Figure 4

The term $\frac{F_1 L}{V_0}$ always occurs in the equation.

Now for a similar example the maximum and minimum pressure are calculated, then the term

$$\left(\text{Log}_e \frac{P}{P_0} + \frac{P_0}{P} - 1\right) \text{ must be equal to:}$$

$$\frac{5000 \times 0.071 \times 1000}{2 \times 9.81 \times 50{,}000 \times 1.2} = 0.3 \quad \text{From fig. 4 one can read}$$

$$\frac{P_{\min}}{P_0} = 0.5, \qquad \frac{P_{\max}}{P_0} = 2.43$$

$$P_{\min} = 25{,}000 \text{ kg. per sq.m.} = 15 \text{ m. head.}$$

$$P_{\max} = 121{,}000 \text{ kg. per sq.m.} = 111 \text{ m. head.}$$

As Equation 4 cannot be solved directly, the period of the pressure wave cannot be calculated.

In fig. 5 is given the curve drawn for the above example, $L = 5000$ m., $S_0 = 1$ m. per sec., $P_0 = 40$ m. head, and $V_0 = 1.2$ cu.m., again what is contained in Equation 4.

Point A represents the working conditions, $\frac{P}{P_0} = 1$. In the beginning, by decreasing the velocity, the pressure in the air chamber is reduced, but this reduction becomes less as the velocity of the center of mass of the water becomes less. When this velocity has become zero (Point B) the pressure in the air chamber has reached a minimum. The water will now flow from the tower to the air chamber, and the pressure in the air chamber increases, the maximum negative velocity occurring when the pressure is again equal to P_0 (Point C). Then the velocity again reduces, and the pressure increases until (point D) the maximum pressure, where the velocity is again equal to zero. From this point the water flows from the air chamber to the water tower and the pressure drops. The maximum velocity is reached at Point A (3.28 ft. per sec. or 1 m. per sec.). Here a new cycle begins, which is theoretically equal to the previous one.

In fig. 5 the dotted line represents the relation between the pressure in the air chamber and the velocity of the center of mass of the moving water, for a volume of air twice as large as in the example above ($V_0 = 2.4$ cu.m.). It reasons that in the example doubling

of the air volume has little influence on the minimum pressure occurring, whereas the maximum pressure undergoes an important reduction.

For the example, fig. 6 represents the relation between the air volume V_0 and the maximum and minimum pressure which can occur by use of these volumes in the air chamber.

We must also note that the size of the air chamber is not conditioned by the volume V_0 but by the maximum air content, V_m, which the air chamber contains when the pressure is $P_{\min}$.

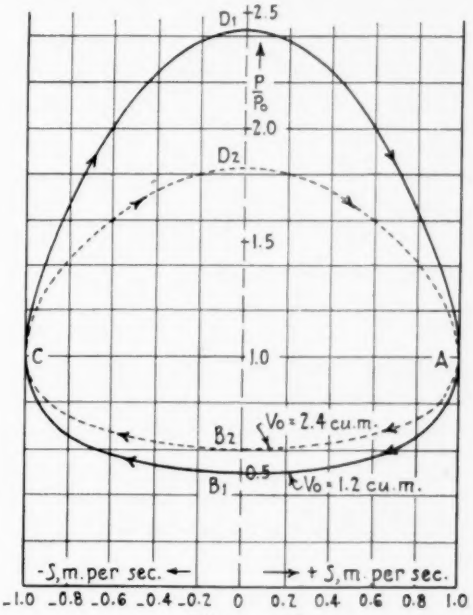

Figure 5

The minimum content of air must be equal to $\frac{V_0 P_0}{P_{\min}}$ because otherwise, air will flow from the chamber to the pipe at $P_{\min}$.

In fig. 6 the relation between the minimum content and the volume V_0 are represented. For example, suppose that the strength of the pipe material used limits the head to 70 m., that is

$P_{\max} = 80{,}000$ kg. per sq.m. and $\frac{P_{\max}}{P_0} = 1.6$; then V_0 must be equal to 3.9 cu.m. whereas the minimum air content must be 5.8 cu.m.

It can also occur that the minimum pressure may determine the size of the air chamber. When, for instance, near the pump branch lines are connected to the main pipe in which the head may not rise above 25 m., that is $P_{max} = 35{,}000$ kg. per sq.m. and $P_{min}/P_0 = 0.7$. The volume V_0 is now 4.8 cu.m. and the last content must be 6.9 cu.m.

Though both the previous methods are based upon the erroneous assumption that liquid and pipe are inelastic, we will show that the applications of these methods in cases where the period of the air chamber is great with respect to $4L/a$ will give usable results, espe-

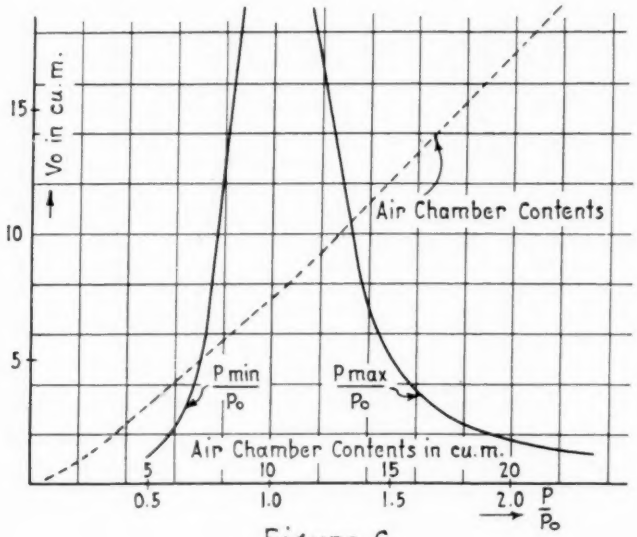

Figure 6

cially the second method when used for the computation of minimum pressures and maximum air chamber content. For this reason they have been treated extensively.

Third Method

The *third method* in which the water and pipe are considered elastic will now be described.

In the previous methods the quantity of water was considered as one compact incompressible mass, and we could consider the movement of the center of gravity of this mass. This is governed by

a force which is dependent upon the distance which the center of gravity moved from the initial conditions.

A simple analogy can be made in which the water is likened to a weight which is hung on a spring. When the weight is moved away the tension in the spring increases; consequently the force tending to restore normal conditions increases. The same analogy holds for a compression of the spring.

In the first method we have a simple spring in which the force is directly proportional to the displacement.

With the second method an imaginary spring was used in which the relation between force and displacement was in the same proportion as the isothermal expansion of the gas.

If we assume that the water and pipe are elastic, we might easily make the analogy that the water mass acts as a second spring. Calling the effect of the air chamber the air spring and the effect of the water mass the water spring, we may combine the effect of these two springs to obtain a picture of the action in a pipe line with air chamber in which the flow is suddenly stopped.

When there is no air chamber on the line, that is no air spring, the pipe line is subject only to the pressure waves arising from the water and pipe line.

If the water and pipe are inelastic, then the pressure in the chamber is subject only to the oscillation which arises out of the elasticity of the air in the air chamber. Combining these two actions, elastic water and elastic air, a vibration arises which is influenced by the peculiarities of both springs.

It will be clear that with a weak air spring (great stretching with little force, large volume V_0) the wave length of the combined vibration will be nearer that of the air spring than when the air spring is stronger (less volume V_0) and the wave length of the air spring approaches that of the water spring. With a relatively small quantity of air, V_0, the water spring will predominate.

The second method of calculation, as may be seen from the above, tends, when the volume of air, V_0, is large, to approach the exact value.

The theory of pressure oscillations developed by Allievi, gives a relation between the reduction of velocity and the reduction of pressure in a pipe line (3). This relation is most easily represented by a graph.

From the theory, if an observer would move with a velocity of

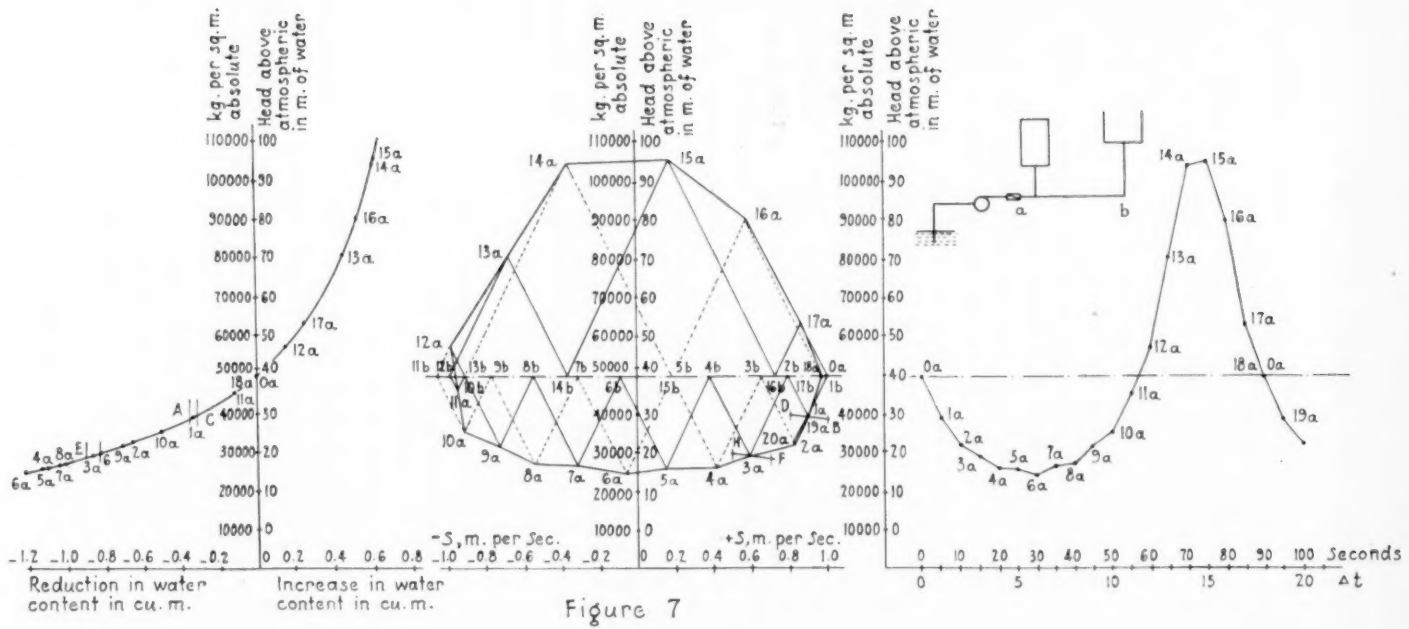
kg. per sq.m. absolute
Head above atmospheric in m. of water
-S, m. per Sec.
+S, m. per Sec.
Reduction in water content in cu. m.
Increase in water content in cu. m.
Seconds
Δt

Figure 7

(a) ft. per sec. through a pipe, he would undergo a pressure variation as follows:

$$P_0 - P = \frac{aw}{g}(S - S_0) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$P_0 - P = -\frac{aw}{g}(S - S_0) \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which equations represent straight lines.

The above law is always valid, hence may be applied in the cases where an air chamber is employed, and will answer the question of which pressure variation will occur. For a graphical solution we can most easily start with a special example.

For this example, the same case is chosen that was used in the first two methods. The velocity of the pressure wave in water is, for simplicity, chosen as 1000 m. per sec.

In fig. 7 the graphical computation of the pressure oscillation is developed. Here all points marked A give pressures and velocities behind the air chamber while points B give pressures and velocities at the standpipe.

The chosen unit of time required for the wave to go from the air chamber to the tower is $\Delta t = 5000/1000 = 5$ seconds.

From Formula 7 we see that pressure reduction is governed by velocity reduction during a certain time period. With the use of an air chamber the velocity reduction is controlled by the quantity of water leaving the air chamber during the time period. Thus, it is first necessary to designate the quantity which flows out of the air chamber during a pressure reduction and flows in during a pressure rise.

In fig. 7 the curve at the left represents the relation for an air chamber with a volume of 1.2 cu.m., for a pressure P_0 of (50,000 kg. per sq.m.) absolute. We take for the construction of this curve that the expansion and contraction of the gas is isothermal, in order to compare with previous methods. We could, however, just as easily use an adiabatic expansion.

When the pump is stopped, the pressure in the air chamber is 50,000 kg. per sq.m. and the velocity in the pipe at that point 1 m. per sec., represented by point Oa in the middle figure of fig. 7. The pressure in the pipe drops after cutting off the motor, just as does the velocity. Between these quantities the relation of Equation 7 exists.

Graphically this relation is represented by line $0a$–$2a$ of which only the point $0a$ and the direction are known. The points $1a$ and $2a$ have still to be found. Assume that the velocity remains constant. Then in Δt seconds the efflux from the chamber will be 1.0 $F_1\Delta t$ cu.m. = 0.355 cu.m. Then the pressure would have decreased to a value represented by point A. This pressure is projected to the vertical line representing 1 m. per sec. and gives point B. Assume that the velocity is reduced to 0.8 m. per sec. then the efflux is $\left(\frac{1.0 + 0.8}{2}\right) F_1\Delta t = 0.32$ cu.m.; the pressure then would be decreased to point C. Projecting C on a vertical line representing a velocity of 0.8 m. per sec. we obtain D. Connect B and D and the intersection of B-D and line $0a$-$2a$ gives point $1a$, which represents the pressure and velocity at a point just behind the air chamber on the moment Δt sec., after the pump stops.

Now if the observer moves with a velocity (a) from the air chamber to the standpipe he then undergoes the pressure and velocity changes according to Equation 6, represented by the line $1a$-$2b$. Point $2b$ is the intersection of line $1a$-$2b$ with the horizontal line representing pressure of 50,000 kg. per sq.m. (the pressure in the standpipe remains constant). The observer now returns with the same velocity, and the line $2b$-$3a$ represents the pressure and velocity changes met on this trip. We must now find point $3a$.

Assuming the velocity to have decreased to 0.7 m. per sec., the efflux in $2\Delta t$ seconds would be $\frac{0.9 + 0.7}{2} \times 10 \times 0.071 = 0.57$ cu.m. and the point E would be found corresponding to F in the middle figure in the same manner as D was found; while assuming velocity of 0.5 m. per sec. points G and H are found. The point of intersection of F-H with line from $2b$ at the known angle gives $3a$.

In the same manner points $4b$, $5a$, etc., are found. Point $19a$ is similar to $1a$ and a new cycle begins.

The second series of points may be found by starting from $1b$. At the moment Δt after stopping the pump the velocity at the standpipe is still 1 m. per sec., because the pressure reduction has not yet reached the standpipe. So points $2a$, $3b$, $4a$, $5b$, etc., are located. It appears that $20a$ does not coincide with $2a$ though it is so situated that the same pressure sequence may be expected. In order not to complicate unduly the figure, the procedure has been carried no further.

In fig. 7 on the right the relation between pressures and time are indicated.

The period is $T = 90$ seconds.

The maximum pressure is 106,000 kg. per sq.m.

Figure 8 shows exactly the same construction for an air chamber with $V_0 = 2.4$ cu.m.; the period, $T = 120$ sec.; $P_{max} = 90{,}000$ kg. per sq.m. = 80 m. head; $P_{min} = 30{,}500$ kg. per sq.m. = 20 m. head.

From table 1 we can compare the values of maximum and minimum pressure by the three methods.

It is clear from table 1 that the errors in the computation by the first and second methods become smaller as V_0 becomes larger. The difference between the second and third methods is also less than between the first and third.

TABLE 1

METHOD	$V_0 = 1.2$ m³			$V_0 = 2.4$ m³		
	Pressure head m.	Min P m.	T	Max P m.	Min P m.	T
First: approx. method	78.5	1.5	83	67.2	12.8	117
Second: exact method	111	15	—	82	20	—
Third: graphical method considering elasticity of water	96	14.5	90	80	20.5	120

In fig. 9 the pressures and velocities computed by the second and third methods are again compared. On the horizontal axis, the velocities near the air chamber are plotted.

The curves of decreasing pressure correspond closely, while the curves of pressures increase diverge more or less as the volume V_0 is smaller or larger. The explanation of these differences is as follows:

The pipe has its own wave length depending on the length and the pressure wave velocity a. If this period is large the divergencies of pressure between the second and third methods will be greater than for a short period (short pipe). As the pressure in the air chamber decreases, there is gradual divergence of velocity and pressure, and the expansion curve makes a smaller angle with the horizontal axis and thus the influence of the wavelength of the pipe is smaller. Because of this the divergencies in the high pressure region are larger.

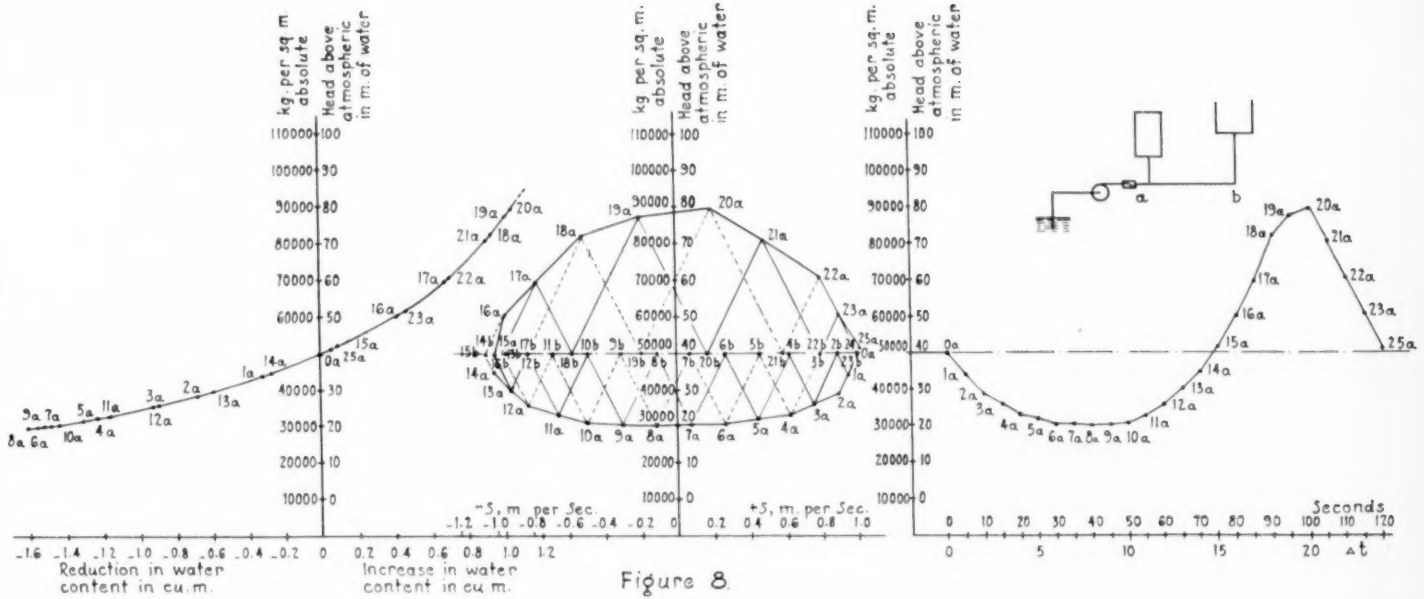
kg. per sq. m. absolute
Head above atmospheric in m. of water
Reduction in water content in cu. m.
Increase in water content in cu. m.
-S, m. per Sec.
+S, m. per Sec.
Seconds
Δt
a
b

Figure 8.

When the variations of pressure are largely dominated by $\frac{V_0}{F_1 L} = \frac{\text{air volume}}{\text{pipe content}}$, the divergencies are, as a consequence of elasticity of water and pipe, under the influence of L/a of the pipe.

What conclusions can be drawn from the foregoing concerning the usefulness of an air chamber as a means of reducing water-hammer pressure rise which occurs upon stopping the centrifugal pump?

With the installation of an air chamber the pressure increases which occur in the pipe are of greater importance than the pressure

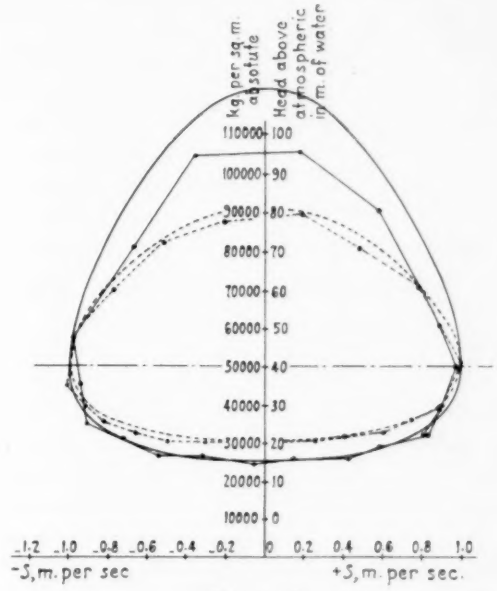

Figure 9

reductions. Also, only use an air chamber to keep the pressure decrease within certain limits. If the chamber has fulfilled this function, put it out of work.

Practically, this solution may be achieved simply by placing a check valve between pipe and air chamber which lets the water pass only in the direction of the standpipe and, as soon as the velocity is zero, closes (see fig. 10).

According to the pressure oscillation theory, the pressure increase can never become greater than the pressure reduction on the moment

of valve closure. In the example above, the pressure is thus never higher than 50,000 + (50,000 − 24,500) = 75,500 kg. per sq.m. = 65.5 m. head for the case of V_0 = 1.2 cu.m. and 50,000 + (50,000 − 30,500) = 69,500 kg. per sq.m. or 59.5 m. head when V_0 = 2.4 cu.m.

Figure 10 represents the pressure wave as it would be when in the above example with V_0 = 1.2 cu.m., a check valve was placed between the pipe and the air chamber. It shows that the pressure varies between P = 75.5 m. and 24.5 m. The maximum pressure which occurs in this case is not more than 59 m. head. This maxi-

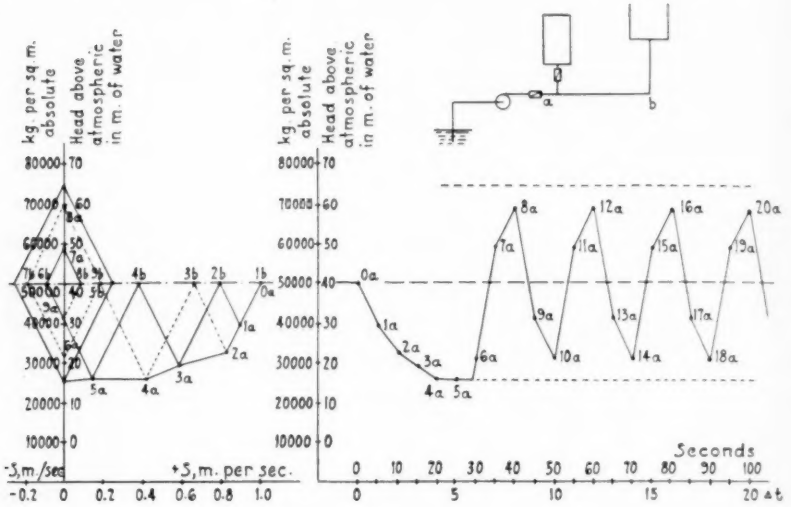

Figure 10

mum pressure is controlled by the location of point 5*b* on the axis P = 50,000 kg. per sq.m.

An air chamber thus can be used as a means of limiting the maximum pressure reduction, and hence the maximum pressure rise.

In order to insure that in a short time after stopping the pump, there prevails in the air chamber a pressure equal to the static pressure, a small pipe, say 1 inch, can be placed parallel to the connection between the pipe and the air chamber. The chamber will gradually refill with water through this pipe and the initial conditions be restored. It is also possible to drill a small hole in the valve for the same purpose, but this is not so efficient.

What would happen if no air chamber was included in the line?

The pressure and velocity in the pipe would decline along the line $0a$-$2a$. It is now of great importance where point $2a$ is located, as this point has great influence on the pressure decrease (on the moment $2\Delta t$ the decreasing wave from the pump returns as a pressure wave from the tower to the pump).

The location of point $2a$ is controlled by:

1. The static head on the pump,
2. The frictional resistance of the pump,
3. The inertia and flywheel effect of the pump.

A complete discussion of these factors would require a great deal of space. The following short discussion must suffice.

The absolute pressure at point $2a$ can never be less than the absolute pressure on the suction side of the pump, from which the pump resistance is subtracted. The pressure will vary with discharge. The point of intersection of this pressure line with $0a$-$2a$ is point $2a$, if we assume that the accumulated energy of the moving parts may be neglected. If the pump resistance is high we can reduce the effect of pump resistance by placing parallel with the pump a bypass of sufficient size (with a check valve with low resistance). The minimum pressure will then be equal to the suction pressure.

Figure 11 shows a pressure curve for such a case for a pump with static head of 40 meters. The inertia of the moving parts is neglected and the pressure will decrease in an infinitely short time from $0a$ to $2a$. The pressures occurring just before stopping are underlined and those occurring just after stopping are not. The maximum pressure rise which can occur under these conditions is equal to the static head on the pump.

If the inertia of the moving parts is considerable, then some time will go by after stopping before the pressure drops to zero. (See Boerendans, *Water*, No. 8, 1938.) Figure 12 shows a possible pressure curve under these conditions.

If, for instance, with an electrically driven centrifugal pump, the connection between the motor and pump is heavy (acts as flywheel) the location of points $1a$ and $2a$ will be greatly influenced.

Conclusions

In general when we stop a pump in a pipe system:

(A) When a maximum pressure rise of $\frac{aw}{g} S_0$ and decrease of same

magnitude are permissible, we need take no measures to limit the pressure oscillations.

(B) An air chamber is unnecessary when it is:

1. permissible to have a pressure equal to the flow pressure plus the static pressure;
2. when a minimum pressure equal to the suction pressure is permissible, because in these cases we can use simpler means to achieve the result.

As applications for the air chamber, there remain those cases where the static head is great. The air chamber then serves to

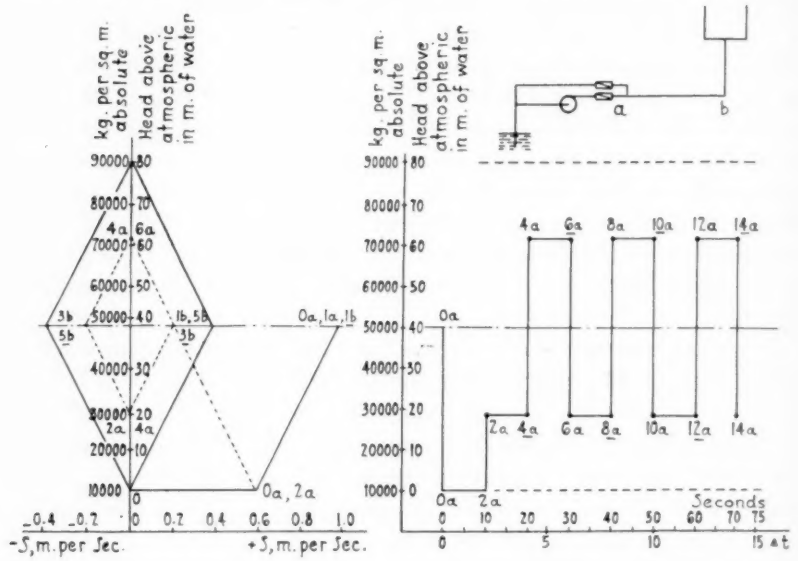

Figure 11

prevent too great maximum pressures, and alone can be used to limit the pressure decrease and should be eliminated when the velocity in pipe becomes negative.

From the above discussion it will be clear why practical experiences with air chambers are so varied, and why so many different opinions are noted. To illustrate this we insert some diagrams taken at the County Water System of Amsterdam.

Before discussing these it will be well to discuss two points.

(A) The assumption of Allievi mentioned above "the hydrodynamical occurrences will also develop according to a law which

possesses the fundamental oscillation period T; these oscillations have superimposed a secondary wave of period $4L/a$. We have then a slow primary oscillation (from the air chamber) and a rapid secondary oscillation (from the elastic wave in the pipe).

From figs. 7 and 8 and again 13, (which represents a point half way in the pipe for the case $V_0 = 1.2$ cu.m.), it seems that the secondary oscillations have a period $2L/a$ not $4L/a$.

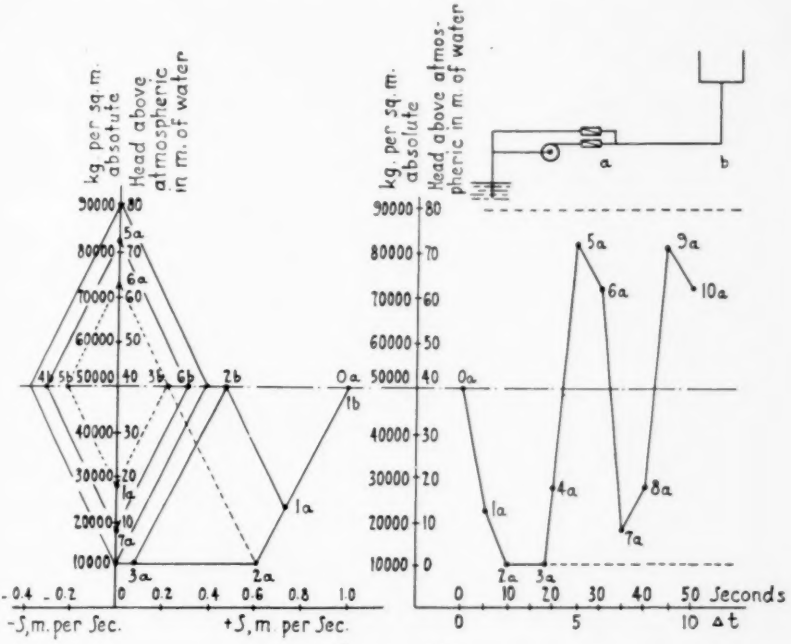

Figure 12

This is easily explained. If there is no air chamber, there is initiated a wave of period $4L/a$. By the initiating of a pressure wave in an air chamber, if the air chamber has a large volume, the chamber will behave as a standpipe. The point where S is zero is now situated not at the pump but midway to the standpipe and the period is $\dfrac{4\left(\dfrac{L}{2}\right)}{a} = \dfrac{2L}{a}$.

(B) When the resistance head against which the pump is working is a great part of the total head on the pump, then the maximum

pressure occurring in the pipe, between pump and standpipe may become considerably lower than the pressure at the pump.

Assume the head on the pump to be $h_m = h_{st} + h_w$ (static head and resistance head) and the pressure at the pump decreases to zero at stopping; then this pressure decrease h_m goes through the pipe and the pressure at the standpipe will decrease to $h_{st} - h_{st} - h_w = -h_w$. This h_w is no longer the original h_w because the velocity has reduced but still it may not be so large that we must take means

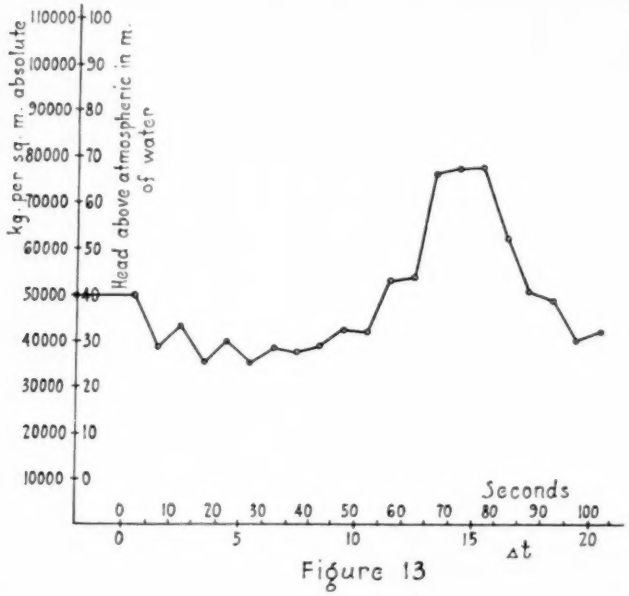

Figure 13

to limit this reduction either by means of an air chamber or by adding inertia to the pump.

With long pipes then, it is not only necessary to compute the effect of pressures at the pump but also the pressures occurring in the pipe between pump and reservoir.

Discussion of Fig. 15

Amsterdam has in the Haarlemmer-meerpolder a pumping station which receives water from Leiduin under head from 12 to 20 meters and pumps the water to Amsterdam under a head of 40 meters.

In the station are two air chambers (fig. 14). Two centrifugal pumps are used. They are rated at 600 cu.m. per hour; one is driven electrically, the other by a diesel engine. The diagrams were taken when the electric pump was stopped. The diesel was idle. The valves *c*, *d*, and *e* in fig. 14 were closed.

The pressures of the water from Leiduin need no consideration as when the pump stops this water is bypassed to a reservoir at Haarlemmerweg. The pipe from the pumping station to Amsterdam is 3,920 meters of 30 cm. cast iron, 3,670 m. of 22 cm. cast iron, 1,648 m. of 18 cm. steel and 582 m. of 22 cm. cast iron which comes into the city system under a pressure head of 28 meters.

With varying volumes of air in the chamber and varying discharge, 50 indicator diagrams have been taken, some of which are shown here. Two different methods of stopping were used.

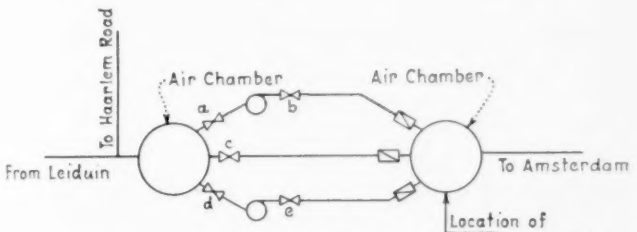

Figure 14

The first series (Nos. 3, 11, 19, 6, 15, 22, 9, 18 and 26) were taken when the flow water was adjusted off by closing valve *b* partially. When flow was established the valve, *b*, was closed as quickly as possible.

In the second series (Nos. 41, 34, 27, 44, 37, 30, 47, 40 and 33) the flow was reduced as before but stopped by shutting down the motor.

The air volumes are based on a head of 28 meters. The oscillations are all damped by friction.

Table 2 gives a review of the maximum and minimum pressures and the period of oscillation. The variations by stopping valve *b* are greater than those caused by stopping the motor. In the second case there is water still flowing through the motor when the pressure drops below the flow pressure.

Diagrams 3, 11, 19, 6, 15 and 22 of Series 1 show the effect of air volume on intensity of pressure very clearly.

Diagrams 9, 18 and 26 of the first series do not show so clearly as with a flow of 600 cu.m. per hour, it was not possible to close the valve entirely within the period of 72 seconds (the wave period of the pipe). Diagram 26 shows this especially well.

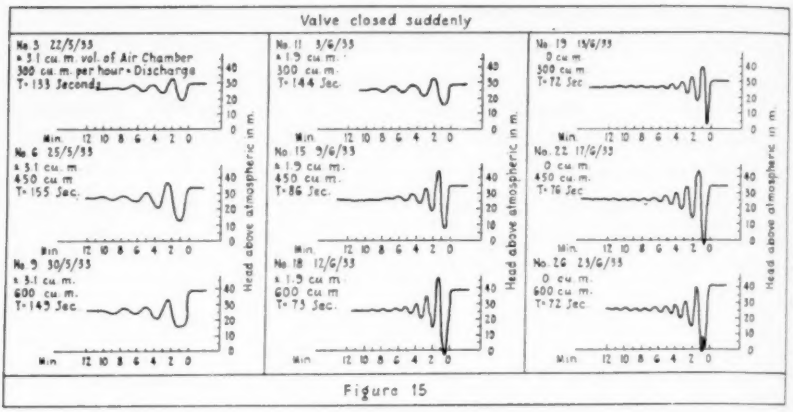

Figure 15

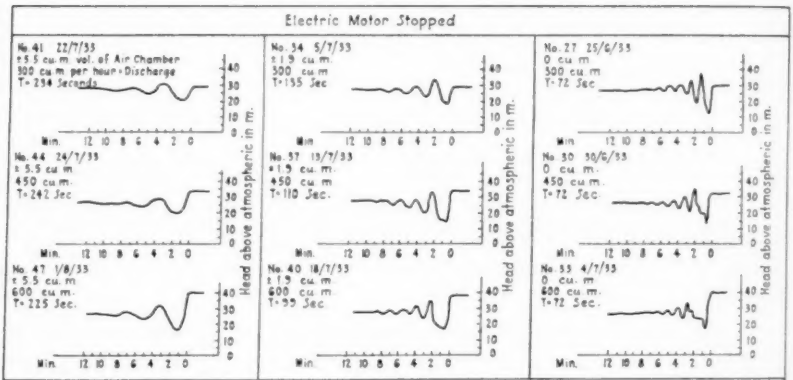

Figure 15

In the second series, the minimum pressure is controlled by the pressure which comes from Leiduin and the pressure increase is always less than the pressure decrease. Only in No. 3 was the period not what was predicted; this may be explained by a difficulty with the indicator motor.

From the diagrams it may be seen that the initial velocity of the water has its influence on the period.

When we compare the pressures by stopping of the motor under ordinary conditions (600 cu.m. per hour) with and without air in the chambers we must conclude that for this station an air chamber is unnecessary.

Higher pressures than that have occurred.

TABLE 2

	OPERATING PRESSURE m.	VOLUME OF AIR AT 28 METERS HEAD					
		3.1 cu.m.	1.9 cu.m.	0 cu.m.	5.5 cu.m.	1.9 cu.m.	0 cu.m.
		Valve shut off			Motor shut off		
300 cu.m. per hr.	29.2	No. 3	No. 11	No. 19	No. 41	No. 34	No. 27
Max. Press. m.		33.3	33.5	40.0	30.8	34.1	38.3
Min. Press. m.		17.6	15.8	1.7	20.0	18.3	12.5
T in sec.		133?	144	72	234	135	72
450 cu.m. per hr.	34.2	No. 6	No. 15	No. 22	No. 44	No. 37	No. 30
Max. Press. m.		37.5	45.0	44.2	27.5	33.0	35.7
Min. Press. m.		12.5	6.2	−5.0	19.2	14.0	13.0
T in sec.		155	86	76	242	110	72
600 cu.m. per hr.	40.0	No. 9	No. 18	No. 26	No. 47	No. 40	No. 33
Max. Press. m.		33.3	46.0	40.5	31.6	35.0	34.0
Min. Press. m.		15.0	13.3	−2.5	15.8	16.2	16.7
T in sec.		149	73	72	225	99	72

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2. HANNEMAN, O. Grundsatze fur Bau und Betrieb der Uberlandleitung der Wurttembergis chen Landeswasserversorgung. Das Gas und Wasserfach, No. 38, (1935).
3. BOERENDANS, W. L. Water (Neth.), Nos. 8, 9, 10, and 11 (1938).

TRANSLATOR'S NOTE: The Boerendans paper is another forward step in the progress of the solution of water-hammer problems encountered in the pumping, distribution and control of any liquid. Unfortunately it was not available prior to the December, 1938 A. S. M. E. Symposium, in which the A. S. C. E. and A. W. W. A. participated. The paper would have provoked valuable discussion and perhaps brought forth further verification of the methods outlined by Boerendans for the determination of proper size of the well

known air chamber widely used to reduce water-hammer pressure surges.

The methods of solution also can apply to closure of valves at the end of pipe lines. The translators have had occasion to check Boerendans' formulae with data taken prior to his publication in Dutch. Data were obtained on a 3-in., 3,000-ft. pipe line discharging gasoline at a refinery where the air chamber contained 2.72 cu.ft. of air. Rewriting Equation 1 so that $m = \rho F_1 L$ (by taking into account density of gasoline) and converting the equation to English units such as lb. per sq.ft. and ft. per sec., the calculated pressure in the air chamber upon closure of valve indicated 166 lb. per sq.in. absolute. The pressure-time diagram showed 169.7 lb. per sq.in. absolute. Calculations by the formulae also agree quite well with the test data shown in fig. 5 and tables 1 and 2 of "Relief from Water Hammer" (Jour. A. W. W. A., **30:** 15 (1938)) where the arresters are obviously not too large or too small.

The graphical method of solution of problems is rapidly receiving wide acclaim. It eliminates approximations and can be made as accurate as construction of graphs permits. Boerendans gives the most condensed explanation of the use of the basic Allievi equations in the graphical analysis of air chamber behavior on pump lines. He undoubtedly has been influenced by Bergeron's contribution to the science.—*L. H. Kessler.*

Water Hammer Studies on Long Pipe Lines

By Laurance E. Goit

THE phenomena of water hammer and the practical significance to the operators of water works are not very widely understood. This condition prevails in spite of the many contributions to engineering literature that have been made by members of the engineering profession on that subject. Theories and formulas developed for computing the effects of sudden changes of velocity in fluid conducting pipe lines are not easily applied to field operations because of lack of understanding of the characteristics of the valves ordinarily controlling flow.

Because of the interest in water works engineering and the development of practical hydraulics taken by H. A. Van Norman and W. W. Hurlbut, respectively Chief and Assistant Chief Engineer of the Bureau of Water Works & Supply of Los Angeles, E. C. Brisbane of the S. Morgan Smith Company and the writer were permitted to conduct tests of water hammer on some of the longest pipe lines in Los Angeles. This opportunity was taken to make tests with the following objects in view:

1. To check water-hammer effects in large pipe lines against the results of studies made on smaller lines.
2. To determine the effects of rate of change of velocity on surge pressures.
3. To determine the effect of rate of closure of various types of valves on rate of change of velocity and on surge pressures.
4. To determine hydraulic characteristics of valves as affected by their mechanical design.

Field tests have been partially completed, and a considerable mass of data accumulated, but a complete analysis of the results cannot

A paper presented on June 13, 1939 at the Atlantic City Convention by Laurance E. Goit, Engineer in Charge of The Distribution Construction and Operation Division, Bureau of Water Works and Supply, Los Angeles.

as yet be presented. This paper is a progress report, describing procedure and equipment used, and commenting on some indicated results that will be of particular interest to water works operators.

The general procedure was to close a valve in a long line that was supplied from an open reservoir, and record all of the pressure and velocity effects. Closures were made at various rates on the following valves:

SIZE	KIND	MAKE
inches		
16	Rotary Cone, hydraulically operated	S. Morgan Smith Co.
20	Rotary Cone, manually operated	S. Morgan Smith Co.
12	Globe, hydraulically operated	Ross Valve Manufacturing Co.
12	Gate, 350 pound hand operated	Rensselaer Valve Co.
12	Gate, 175 pound hand operated	Rensselaer Valve Co.
16	Gate, 175 pound hand operated	Rensselaer Valve Co.
24	Gate, 175 pound hand operated	Rich Manufacturing Co.

Tests were made at convenient times to avoid peak loads. Recording pressure gages were connected at the selected places on the 20-mile length of a line and a man stationed at each to make time marks on the chart. The quantity of water flowing through the line and the valve to be operated were regulated with another valve just downstream from the test valve. Instruments were used to record all pressures, quantity, and movement of valves. Water hammer was produced by closing the test valve at various speeds. Several closures at uniform speeds were made, and then closures at fast speed for the first part and slow speed for the last part were made. Eighty-four valve tests have been made.

The most important pressure recordings were made with strip chart recorders with a speed of one inch per minute. Five such instruments were used with maximum pressure ranges of 150 to 750 pounds. Use was also made of six round chart recorders with special clock works giving one revolution per hour. A special mechanism was built onto one of the strip chart recorders to get, in addition to the pressure recording, a graphic record of valve movement. Another strip recorder had the pen actuated by the venturi meter differential float giving a graphic record of changes of flow in the pipe.

After assembling the necessary instruments, a serious problem was

the co-ordination of records being made simultaneously, but scattered over twenty miles of city. The observers' watches were carefully set and checked, and time marks were made every ten minutes on the charts by the observer. It was found that this left much to be desired because of human errors, but principally because of varying rates of the instrument clock works and of the men's watches. In an attempt to reduce these errors, several special tests were made for the primary purpose of obtaining accurate timing. Several electric clock motors were obtained having a spindle rotation suitable for attachment directly to the driving roll of the strip chart. Electric power connections were obtained from houses, in one location over five hundred feet of wire being run, crossing Sunset Boulevard by suspending the wire from trees on each side of the highway. A chart speed of 3.6 inches per minute was obtained. Time signals were obtained from the Police Department radio, five special radio receivers being used. Altogether about $6,000 worth of portable instruments were used.

The 16-inch rotary cone and the 12-inch globe valves were automatic pressure regulators. The automatic controls were removed and special piping assembled for hand control. The other valves were manually operated, and a special setup of an air motor was made to operate these.

The recording instruments were connected up and started at a predetermined time when all branches were shut off and consumers' water supply rerouted. The flow in the pipe line was adjusted to the desired velocity by throttling with a valve downstream from the valve to be used for the test. Operations of the test valve were then made at various speeds, stopping the set flow and creating water hammer.

The hydraulically operated globe valve and cone valve were not difficult to handle, a little practice making it possible to get reasonably smooth movement. Also the 20-inch manual rotary cone valve and 16-inch gate valve were handled by the air drill motor with fair results. The 12-inch gate valve was not geared so it required too slow turning for the air motor and was operated by men walking round and round. The great increase in torque required for the last few turns made it difficult to maintain a uniform rate of rotation, but results seem to be usable for analysis.

The 12-inch 350-pound gate valve could not be shut by hand more than about 75 per cent of the stroke, and then could not be opened

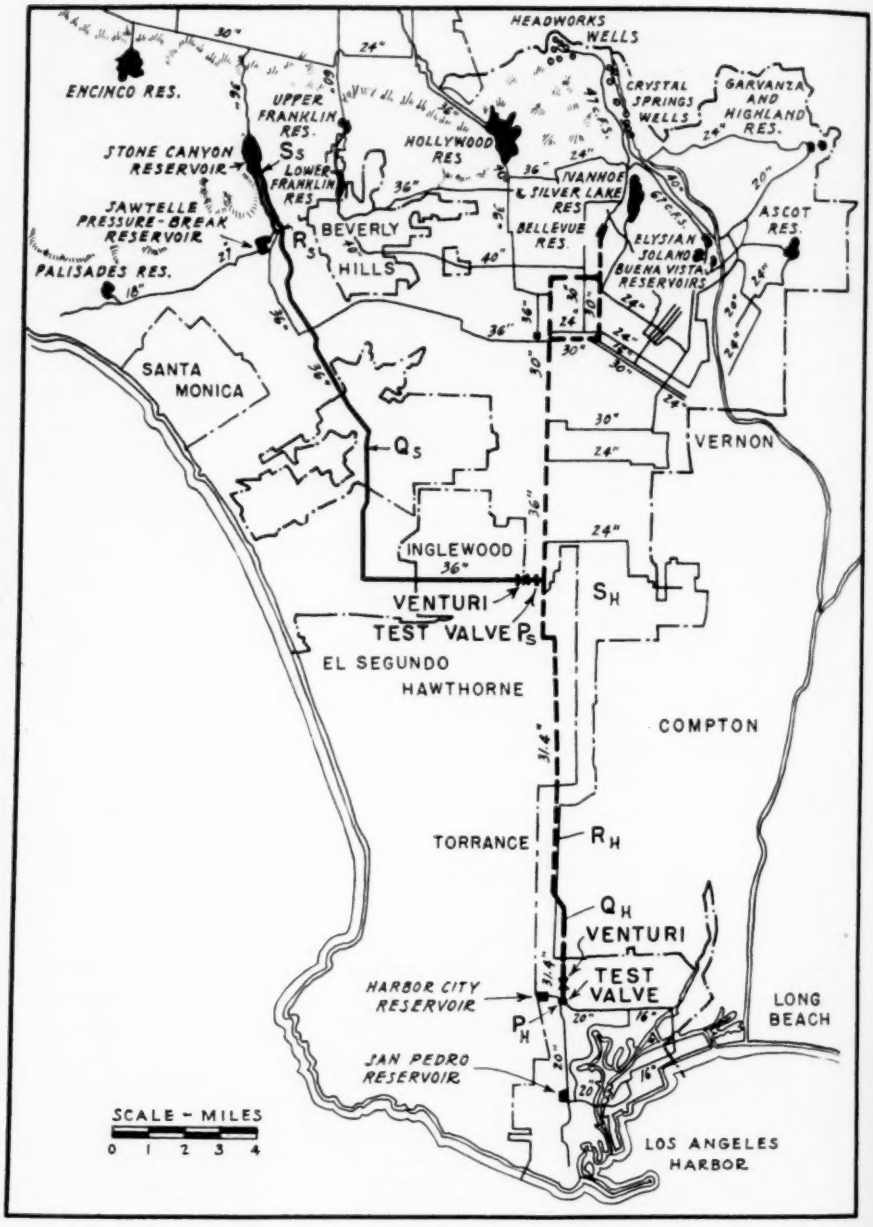

FIG. 1. Map Showing Pipe Lines on Which Water-Hammer Tests were Conducted

without practically balancing the pressure on both sides of the valve. Operation of this valve was abandoned. The 24-inch gate valve nearly proved to be another Waterloo. The air motor could run the valve down to about 90 per cent shut, and there we were stuck. Several trials were made, but as soon as a portion of the flow was cut off and a differential pressure built up the torque required increased very rapidly.

The map, figure 1, shows the skeleton of the distribution system of Los Angeles, and indicated thereon are the two pipe lines on which tests have been conducted. The Stone Canyon—Western Avenue line consists of 94,399 ft. of 36-inch welded steel pipe varying in

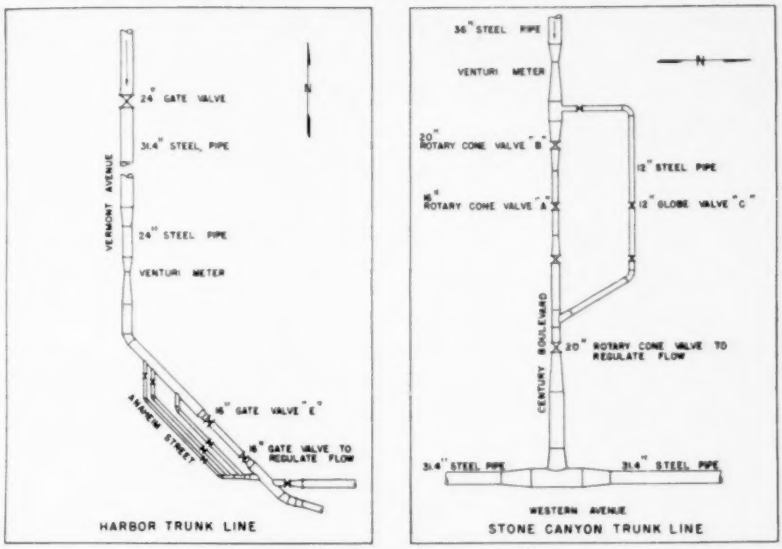

FIG. 2. Terminal Connections of Both Lines

thickness from $\frac{3}{8}$ to $\frac{9}{16}$ in. It was possible to isolate this line by shutting off all branches except three that were equipped with pressure reducing valves operating at such high differentials that the surge waves could not pass through. The Harbor Trunk line with a total length of 114,948 ft. consists of sections of welded steel, riveted steel, and cast iron with diameters of 30 to 36 in. Since this line traverses the main part of the city, branches could not be shut off and secondary waves affected the shape of the recordings. Recording pressure gages were distributed along each line as indicated on the map.

The terminal connections of each line are shown in fig. 2. There is a venturi meter on each, and valves of different kinds.

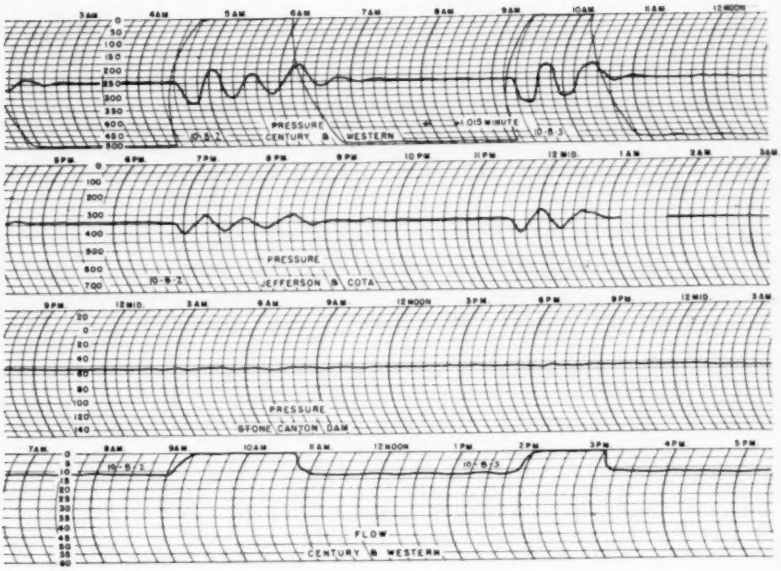

FIG. 3. Group of Original Records, with Time Lined Up

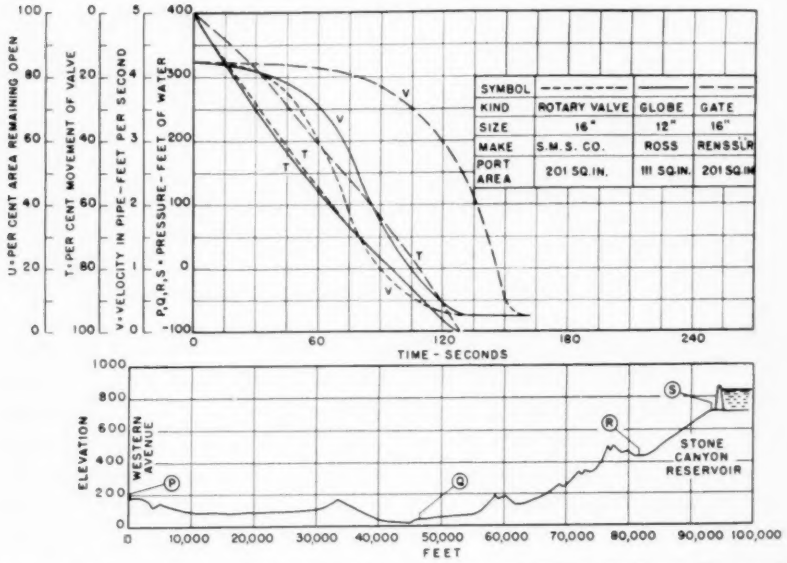
U - PER CENT AREA REMAINING OPEN
T - PER CENT MOVEMENT OF VALVE
V - VELOCITY IN PIPE - FEET PER SECOND
P,Q,R,S - PRESSURE - FEET OF WATER
TIME - SECONDS
SYMBOL
KIND ROTARY VALVE GLOBE GATE
SIZE 16" 12" 16"
MAKE S.M.S. CO. ROSS RENSSLR
PORT AREA 201 SQ.IN. 111 SQ.IN. 201 SQ.IN
ELEVATION
WESTERN AVENUE
STONE CANYON RESERVOIR
FEET

FIG. 4. Composite of Various Kinds of Valves at Same Uniform Time of Operation

A group of the original records is shown in fig. 3 with the time marks lined up. For study and comparison these recordings are being transferred to rectangular co-ordinates, putting them all on the same scale and eliminating the effect of elevation. On this record sheet, shown in fig. 4, the zero pressure point represents the pressure at the lower end of the test line with the set velocity flowing.

This paper will not deal with the mathematical aspects of water hammer, concerning wave velocity and pressure rise; but instead will present some composite charts from these tests and point out a few practical lessons in valve operation that can be drawn from this work in its present state.

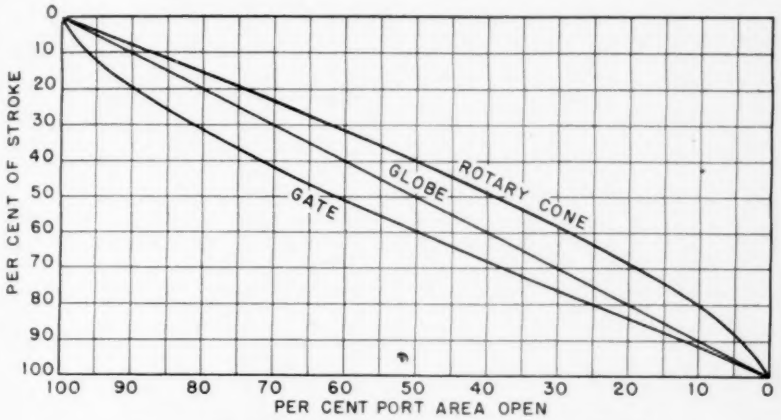

FIG. 5. Curves of Valve Openings—Area vs. Stroke

Figure 4 is a composite of recordings from the operation of various kinds of valves, shut at uniform rate of stem movement, and in each case stopping the same velocity in the pipe line. Examination of the velocity curves shows that appreciable reduction of velocity in the pipe line does not occur until the valves are partly closed. The rotary cone valve affects the velocity earliest. The globe valve is next in this respect, and the gate valve affects the velocity very late in the stroke and stops all of the water in the last few per cent of stroke. These points of noticeable velocity reduction become earlier with increased quantities of water, but the relation regarding style of valve remains.

The reason for this difference is seen in the curves in fig. 5. These

show for gate, globe, and cone valves the area of port remaining open at various points of the stroke of the stem, or in the case of the cone valve, rotation of the plug. In the globe type of valve the area of the port is directly proportional to the stroke and we have a straight line. In the gate valve, the area is reduced slowly at first as the disk passes over the opening, and the rate of area cutoff increases as the stem is turned. In the cone valve the reverse is true, the area of

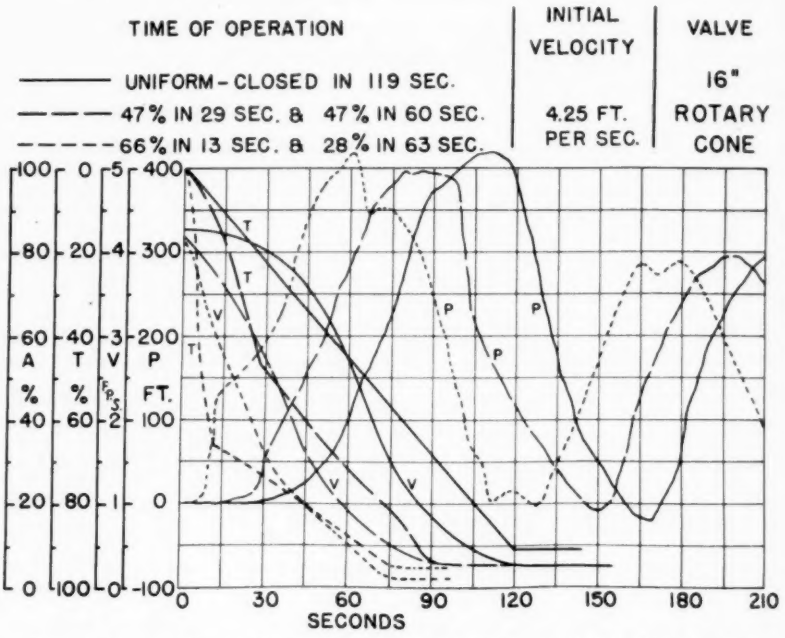

FIG. 6. Composite of Cone Valve Tests at Three Variable Speeds. Same pressure rise from three different total times, because of variable rates of operation.

the port reducing rapidly at the start of closing and the rate diminishing all the way to complete closure.

It is apparent that the first part of the closing of any of these valves could be made more rapidly than was the case in the tests shown in fig. 4, with the rate of operation slowing up toward the end. In fig. 6 are shown three curves of operations of a 16-inch rotary cone valve using differing rates of movement. With approximately uniform movement, the time consumed in closing the valve was 120 seconds. When closed rapidly for almost half the way and then fin-

ished slowly at almost the same rate as the first uniform operation the time taken was about 90 seconds. In the third test shown two-thirds of the movement was made rapidly and then movement slowed up to finish closing, and 75 seconds were consumed. The pressure rise due to water hammer is shown for each, and all are of about the same magnitude.

Time in closing a valve can be saved without raising the water-hammer pressure by varying the speed of operation; but how to know what variations of speed to use then becomes the question.

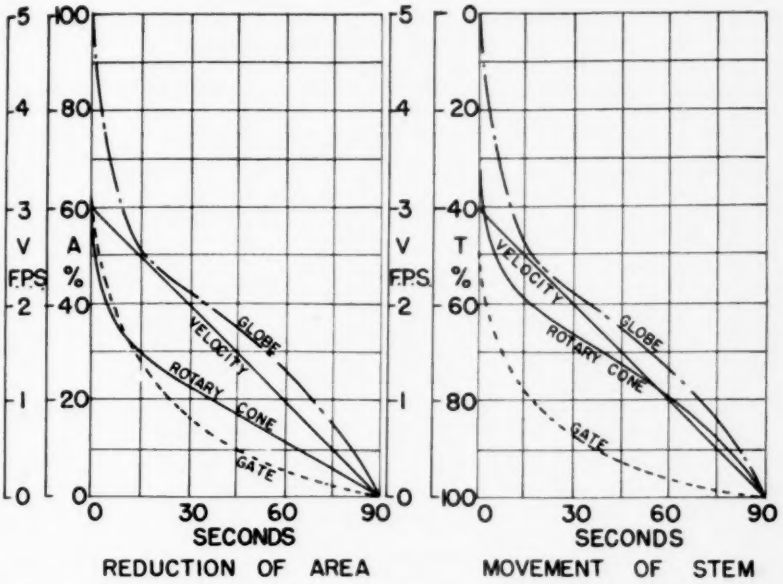

FIG. 7. Ideal Operating Curves to Produce Ideal Velocity Reduction

The ideal line of velocity reduction to cause the least water hammer under a given set of conditions would be a straight line on fig. 6. This ideal condition was approached in one of the curves on this figure when two rates of operation were used.

Starting from this ideal line of velocity reduction, and using the characteristics of the various valves obtained by analysis of and interpolation from many charts, there have been built up hypothetical curves for rate of operation of the valves to produce this ideal velocity reduction, as shown in fig. 7. If each type of valve—gate, globe, or rotary cone—could be moved according to this chart, each would

reduce velocity at the same straight line rate and the water-hammer pressure rise would be the same for each.

On the left half of this chart are curves of area reduction versus time, all showing the same total time of 90 seconds used in making the shutoff. We might expect that the port area reduction curve would be the same for all, but each kind of valve represents a quite different kind of orifice.

The stem movement curves on the right are made by correcting the area curves according to the curves of fig. 5.

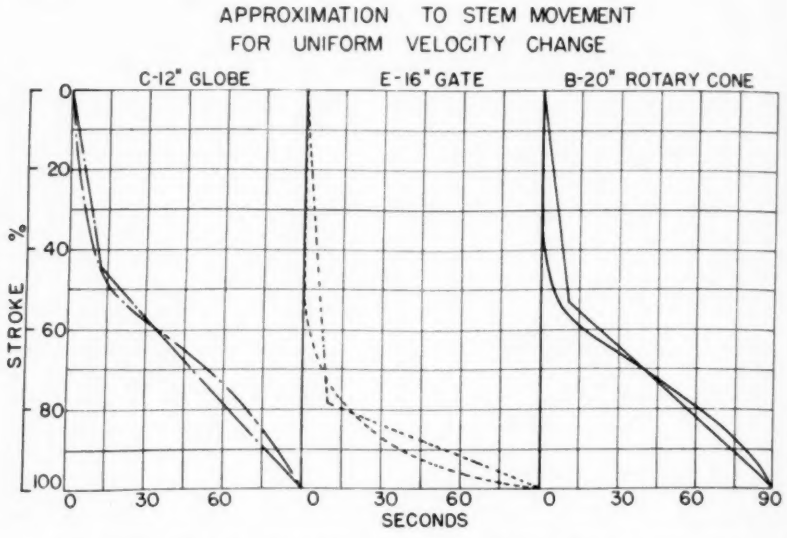

FIG. 8. For three kinds of valves, approximation to ideal to be accomplished in one fast and one slow part.

Water works are full of valves that have to be operated without the benefit of stems with variable thread lead, so there has been constructed a set of curves (fig. 8) showing a good approximation of the ideal curves, where the valve stem is operated fast for part of the stroke and slow for the balance. These curves are based on a velocity in the 36-inch pipe line of 3 feet per second. The higher velocities that usually flow in the trunk lines would probably require operation at the fast speed for a smaller percentage of the stroke, and also to produce no higher water hammer in the line a longer total time must be taken for complete closure of the valve.

Figure 9 is a reproduction of a family of test curves using a 20-inch rotary cone valve. The time of each closure is approximately the same, the variable being the original velocity in the pipe line. The pressure rise curves show the effect of initial velocity on water-hammer pressure. There is a tendency with all of the valves to cause

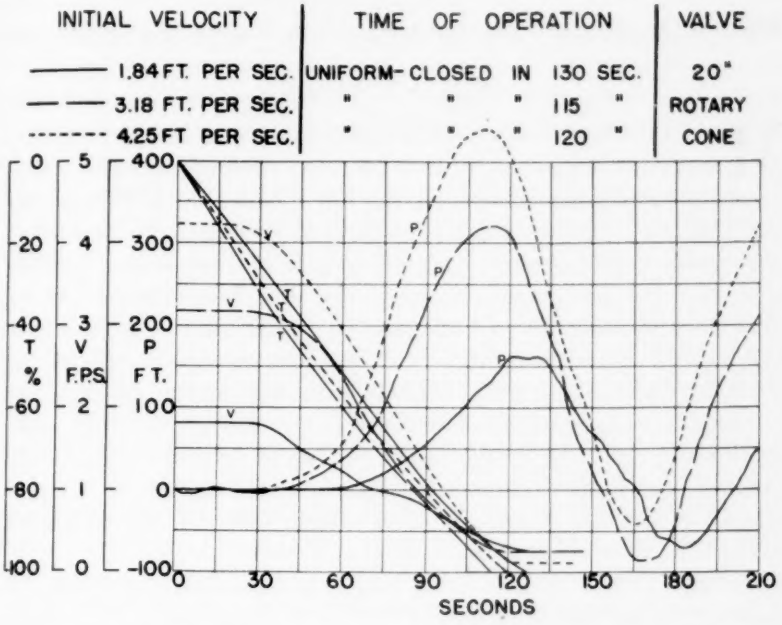

FIG. 9. Three curves at uniform rate, various Q, showing effect of Q on reduction of V curve.

earlier reduction of velocity with increased initial velocity, though this effect is not clearly shown in the curves presented in this figure.

Summing up the results of this partial study of data, the author finds that time of shutoff of a long pipe line can be greatly reduced through proper design and operation of valves and that the resulting pressure rise can still be kept within safe limits.

Water-Hammer Control by Proper Valve Installation

By E. C. Brisbane

MR. GOIT has discussed the characteristics of various types of valves as evidenced by data obtained through exhaustive field tests. A careful examination of the test records brings to light the fact that, in effect, no valve of present day design embodies all of the features necessary for perfect control of pipe lines. Certain types of valves more closely approach the ideal design than do others.

In the first place, a trunk line may be a force main or gravity supply, void of laterals. Or it may be installed to supply various districts by combining a long flow line with distribution or any other combination that conditions demand. The topography may make it necessary to cross rivers, bayous or areas subject to flood conditions. Lines may traverse heavily populated areas, yet irrespective of design, size, length, head or terrain, there is ever present a variable number of hazards.

Probably the least commonly understood hazard is the effect of water hammer upon the lines.

In the studies on water hammer or rather as a part of them, efforts have been made to determine whether any valves now designed could be made to operate in so short a time as to create a hazard. Ten years ago, the author sometimes informed operators or engineers that a theory had been developed which indicated, and it had been proven to a small extent, that by proper timing of a valve, surges could be brought under control.

However, in most cases, when it was explained that extremely rapid cutoffs were to be made as a function of surge control, many an old-time operator turned several shades whiter, and had a few of us been subject to discouragement, the advance and development of today would probably have been seriously retarded.

A paper presented on June 13, 1939 at the Atlantic City Convention by E. C. Brisbane, Chief Engineer, Valve Division, S. Morgan Smith Co., York, Pa.

Credit for the present day developments is largely due to such men as Gayton of Chicago, Hurlbut of Los Angeles, Howson of Chicago and a handful of other engineers and operators. Some had faith in the relatively new theories of valve operation and others, through desperation resulting from trouble, were willing to try anything that offered the possibility of safer operation.

A major portion of the serious difficulties encountered in earlier days were created through the advent of the centrifugal pump. In recent years a cycle of aqueduct or long gravity pipe line building has again increased the many problems involving the design of controls and operation of valves.

The importance of rapid valve operation is more pronounced in the control of pipe lines than is generally realized. It is hoped to develop later a detailed description of the relationship between time and pressure rise.

Lines may be ruptured as the result of floods, settling, earthquakes, electrolysis, abnormal fatigue or faulty installation. A factor affecting or aggravating many of these emergencies is represented in the type, design and operation of the valves installed. All lines should be provided with a sufficient number of valves located at strategic points to facilitate isolation of the line at hazardous locations and in the fastest possible time. Lines may be partially protected by the proper location of emergency check valves, so designed that they will close automatically upon rupture. The increased velocity as the result of the rupture automatically causes the valves to close.

These valves should be designed to cut off the flow in a time considerably shorter than the $2L/a$ of the line (L = length of line; a = velocity of pressure wave). It is often found practical to install a battery of relief valves, located at or near the emergency check valves, to relieve the sudden rise of pressure caused by fast closure. This precaution should be taken provided the design of the line is such that it would be impractical or hazardous to impose pressure on the line equal to the critical or maximum pressure possible under maximum flow conditions.

If the line is 16 in. or over, a battery of small relief valves adjusted to the various amplitudes of the line is apparently more satisfactory. Manual stop valves should be installed at intermediate locations for normal operation. The number and location depends upon the design, the profile and the present and future requirements of the line.

Pressure reducing valves also are commonly required for line operation. The controls of these valves as well as the controls of the emergency check valves should be provided with a wide range of adjustment for independent timing of the opening and closing speeds. The range of adjustment of the pressure reducing element should also be wide. If the valve is an emergency check valve, then there should be a reasonable adjustment for the control of the velocity range through which it may be required to operate.

Pressure reducing valves operating in throttled positions for long periods of time where velocities through the valves are high should

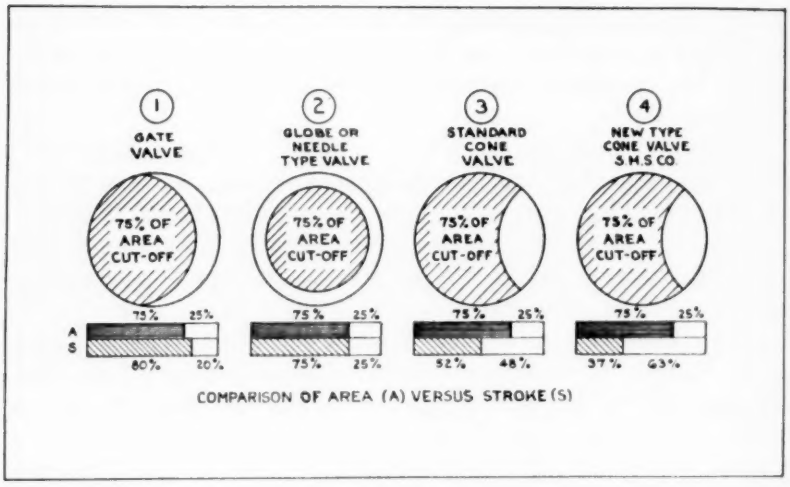

FIGURE 1

be provided with some method of vacuum relief covering those areas that are subject to cavitation.

Since it has been found that fast rates of valve operation may easily be accomplished and are often desirable, the following factors become a pertinent part of valve design:

1. The characteristic of the valve with relation to per cent of area reduction versus time or stroke.

2. Operating torque necessary to overcome the hydraulic unbalance plus the mechanical or sliding friction and the operating torque correlated with time.

Figure 1 shows section characteristics of the various types of valves commonly used. It gives a direct comparison of the cut-off

section in various types of valves. The shaded section represents 75 per cent of the area cut-off and the light section the remaining open area. Below will be noted the per cent of stroke required to reduce the area 75 per cent and the per cent of stroke left to cut off the final 25 per cent of area.

It will be noted that the gate valve utilizes 80 per cent of stroke for 75 per cent of area reduction leaving 20 per cent of stroke for the final 25 per cent of area cutoff.

The globe or needle type valve cuts off 75 per cent of the area with 75 per cent of stroke, leaving 25 per cent of area to be cut off in 25

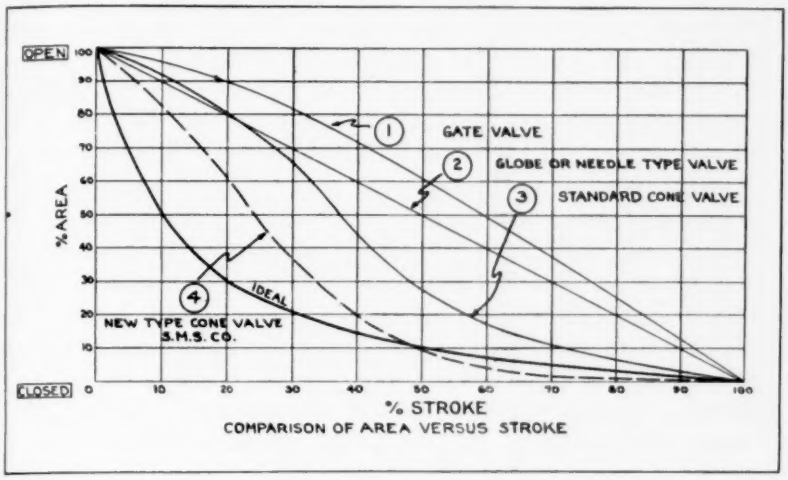

FIGURE 2

per cent of stroke. This is represented as a straight line movement of area versus stroke.

The standard cone valve cuts off 75 per cent of area in 52 per cent of stroke, leaving 25 per cent of area to be cut off in the remaining 48 per cent of stroke.

The No. 4 valve in fig. 1 is a cone type valve with a modified variation in plug movement bringing the cut-off curve closer to the ideal. In this case, 75 per cent of the area is cut off in 37 per cent of the stroke, leaving 63 per cent of the stroke for the last 25 per cent of area reduction.

Figure 2 is made from a compilation of the data and findings resulting from the tests recently made in Los Angeles. Mr. Goit in his paper in this issue of the Journal (figs. 7 and 8) showed what appeared

to be an ideal curve of cut-off for each type of valve. Comparison curves are, therefore, made of all types of valves of present day design showing per cent of area versus per cent of stroke with an ideal curve of area reduction versus stroke or time. This does not represent the ideal curve for any particular type of valve, but is the approximate general ideal from a standpoint of cut-off versus pressure rise and time.

This ideal curve, of course, cannot be 100 per cent accurate for all sizes of valves or all rates of flow, but it does strike a medium well within the limits of safe operation in conjunction with faster closing times than have previously been considered safe.

The writer's organization has contributed both financially and technically to this research effort and has also cooperated with many cities in endeavoring to understand these problems more thoroughly. Extensive hydraulic and mechanical tests have also been made in their own hydraulic laboratories. They have been able to develop mechanical operation in their design of cone valves that will produce a cut-off curve more closely approaching the ideal. The heavy dotted line shown in fig. 2 represents the curve of this new development.

It is not to be construed that the writer's organization is the only one which can produce valves having this characteristic. It has been accomplished by them through a close working knowledge of actual conditions. Once all of the test data become available and public, other valve manufacturers will be able to modify their valve design to conform more closely with the actual operating requirements.

The Correlation between the Carbon Dioxide and Mineral Content of Water and its Corrosivity

By Willem Rudolfs and T. T. Wong

A REVIEW of a number of studies, investigations and theories concerned with underwater corrosion leaves the impression that several interesting questions remain unanswered or may well be reconsidered.

Studies in progress since June, 1931, in the Department's laboratory, dealing with correlations between the geology of water bearing strata and the mineral constituents of deep well waters, between the mineral constituents and degree of corrosiveness of surface and well waters, the relation between mineral constituents and types of scale formation, and the effect of certain types of treatment on corrosion and scale formation, have produced some results which do not always fall in line with the current theories, or should be emphasized to strengthen some of the suppositions.

These laboratory results on the relation between mineral constituents of certain surface and deep well waters found in New Jersey and their corrosiveness, are presented to stimulate discussion and further study.

Materials and Methods

For correlating mineral constituents and relative corrosiveness of water, samples of water containing a wide range of different minerals were collected and their corrosiveness determined.

Surface waters included water from the Pequest, Allamonchy,

A Journal Series paper, New Jersey Agricultural Experiment Station, New Brunswick, N. J., presented on June 15, 1939, at the Atlantic City Convention by Willem Rudolfs, Chief, Dept. Water & Sewage Research, and T. T. Wong, then Graduate Student, Dept. Water & Sewage Research. Mr. Wong later became chemist at the Shanghai water plant. Part of this work was done with the aid of the W.P.A.

Musconetcong, Raritan and Rancocus rivers and some natural spring waters. Also waters from deep wells (varying from 150 to 300 ft. deep) located in the different parts of the state were collected. The location and geological formations will be described elsewhere.

Analyses of the samples of water included dissolved oxygen, carbon dioxide, pH, hardness, total acidity, mineral acidity, total alkalinity, carbonates, bicarbonates, sulfates, chlorides, nitrates, sodium, potassium, calcium, magnesium and iron. Carbonates and bicarbonates were calculated from alkalinity determinations and hardness from the weight of calcium and magnesium. Analyses were made in accordance with standard methods, except for some determinations where minor modifications had been shown to produce better and more consistent results.

As an index for the determination of relative corrosiveness a number of methods suggested in the literature were tried. Since we were not particularly interested in the absolute amount of corrosion, but in the relationships between the different chemical constituents and degree of corrosion, as well as the relative activity of the most active constituents, the method used as index was considered relatively unimportant. Because the corrosiveness is in general proportional to the exposed surface, uniform, clean iron wire was obtained from a chemical supply house and adopted throughout the series of experiments.

Immersing coils of iron wire in water allows the formation of a protective coating which forms faster than under practical conditions. Continuous shaking causes parts of the protective coating to become dislodged, but gentle movement of the water over the iron surface brings different portions of water in contact with the iron surface, without disturbing the coating and resulting in good checks when repeated.

Pieces of iron wire, cut 10 in. in length, were cleaned with fine sand paper, washed with distilled water, followed by alcohol and ether, wiped dry, weighed and placed in a desiccator to prevent attack by moisture and gases. The cleaned and weighed wires were placed in a series of large tubes filled with the water samples supported by means of two pieces of thin rubber to prevent touching of walls and to give uniform exposure, and the tubes were closed by rubber stoppers. The loss of weight of the wire and the total iron in solution were determined at intervals according to the following procedure: After eight hours of immersion, the wire in one tube was taken out of

the tube and the corrosion products removed by the sodium hydroxide method suggested by Finnegan and Corey (1). The wire was treated with 20 per cent hot sodium hydroxide solution to which powdered zinc had been added; after one hour of treatment the wire was washed in distilled water, immersed in alcohol and ether, dried and placed in a desiccator for one hour and weighed immediately. The iron in solution was determined in accordance with standard methods of water analyses. For determination of continued corrosion rates, the water was replaced and the procedure repeated daily for eight days. The dissolved oxygen, free carbon dioxide and pH values were redetermined for each sample of water.

For comparison some corrosion rates of various sulfate, chloride and bicarbonate concentrations were determined by the same methods used for natural waters. Chemically pure sodium sulfate, sodium chloride and sodium bicarbonate were dissolved in boiled distilled water.

Still other comparisons were made where natural waters were subjected to vacuum to remove part of the oxygen and carbon dioxide, while to other portions of the same well waters small quantities of carbon dioxide were added.

Results

The chemical analyses of a group of surface and deep well waters are given as examples in table 1. An analysis of a spring water is included as an illustration of high oxygen, carbon dioxide and nitrate content.

The surface waters varied from very acid to rather alkaline conditions, while the well waters varied from neutral to somewhat alkaline. The free carbon dioxide content varied from 0.2 to 22 p.p.m., while bicarbonates varied from zero to over 200 p.p.m.

Examples of the corrosiveness of a group of surface waters containing varying combinations of chemicals are given in fig. 1 and of a group of well waters in fig. 2. The rate of loss of weight corresponds rather well in the surface waters with the rate of increase of iron in solution. With some of the harder well waters the amount of iron found in solution is less than in the softer waters, irrespective of the quantities of dissolved or free carbon dioxide. While the rate of loss of weight may be greater in such waters, the rate of solution of iron is less. This does not necessarily mean that the rate of corrosion is less. The decrease in the rate of corrosion with time is due to the

formation of a coating or film, protecting the submerged body from direct contact with the corroding medium.

The rate of corrosion in well waters as compared with the iron in solution varies to a greater extent than in surface waters. Attempts to correlate the various single mineral constituents in waters and their corrosiveness were not very satisfactory when all waters were

TABLE 1

Chemical analysis of natural waters as received (in p.p.m.)

Source	Streams					Deep wells						Spring
Number	1	3	6	7	10	2	4	8	9	11	12	5
Dissolved oxygen	2.8	4.0	4.9	8.5	11.0	5.8	10.2	1.8	4.2	3.2	2.1	11.2
Total acidity ($CaCO_3$)	3.3	2.6	5.0	4.0	12.0	3.9	4.7	4.5	5.5	4.5	6.5	7.0
Mineral acidity	0.0	0.0	0.0	0.0	1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
pH	7.2	7.5	8.0	8.4	4.6	7.3	6.9	7.3	7.6	7.0	7.7	6.2
Total alkalinity ($CaCO_3$)	21.6	23.5	137.5	54.0	—	63.9	77.0	115.4	88.2	72.0	178.0	26.5
Normal carbonates	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Bicarbonates	26.4	30.0	167.8	65.9	0.0	78.0	93.9	140.8	107.6	87.9	217.2	32.3
Free carbon dioxide	3.0	4.6	4.5	3.3	5.2	8.8	13.5	10.0	2.3	14.5	3.7	16.2
Sulfates	34.7	23.1	43.8	30.9	23.5	12.2	19.8	360.8	67.2	17.4	42.1	17.4
Chlorides	10.2	6.3	11.5	10.5	11.0	12.6	19.5	29.5	14.0	13.5	12.5	60.4
Nitrates	0.5	1.2	1.2	0.8	0.7	1.4	0.5	1.8	0.9	0.2	2.7	28.3
Iron	0.2	0.2	0.1	0.2	0.4	0.2	1.6	0.2	0.3	1.6	0.2	1.2
Calcium	10.3	7.1	23.7	13.1	1.7	18.3	18.1	62.7	19.2	23.0	36.8	16.5
Magnesium	3.3	3.5	19.8	4.1	0.7	3.7	14.2	56.8	15.9	3.2	14.3	18.0
Sodium and potassium	9.6	9.2	28.5	22.2	8.4	5.8	8.7	84.7	46.4	11.2	17.9	13.3
Calculated hardness	39.3	32.1	140.4	49.6	7.1	60.9	103.6	389.6	114.2	70.7	150.6	115.1

considered irrespective of their sources. Examples of the chemical constituents and some hypothetical combinations and the corrosiveness after 8 days are shown in table 2. The examples given were chosen to indicate that in natural surface and well waters: (1) more than one factor may be effective in corrosion, (2) the same type of salt may affect corrosion differently, and (3) the combination of dissolved salts may produce different effects.

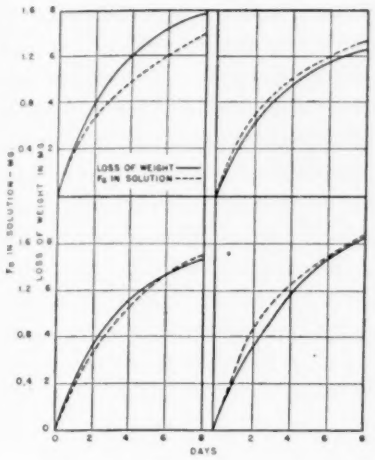

FIG. 1

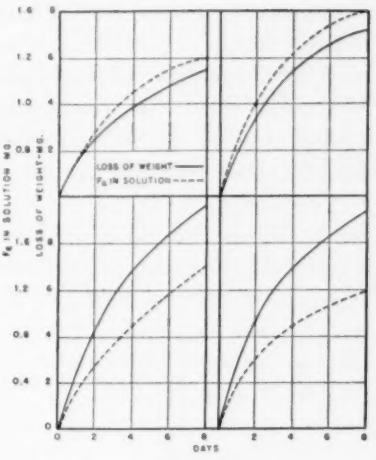

FIG. 2

FIG. 1. Rate of Corrosion and Iron in Solution in Surface Waters
FIG. 2. Corrosion and Iron in Solution in Well Waters

TABLE 2

Relation between minerals and hypothetical combination and corrosiveness

NUMBER	LOSS OF WEIGHT	MINERAL										HYPOTHETICAL COMBINATION			
		D.O.	CO_2	pH	Ca	Mg	Na	SO_4	Cl	$-HCO_3$	$NaNO_3$	NaCl	Ca, Mg, Na Sulfates	Mg, Ca, Na Bicarbonates	Total Diss. Salts
Surface															
10	8.4	5.7	4.6	4.4	1.7	0.7	8.4	23.5	11.0	0	1.0	18.1	12.4	0	49.5
1	7.9	6.8	1.4	7.2	13.3	3.3	9.6	34.7	10.2	26.4	1.6	16.8	47.2	16.2	81.1
7	7.4	5.3	2.5	7.6	13.1	4.1	22.2	30.9	10.5	65.9	1.1	17.3	46.6	77.6	142.9
3	6.3	5.2	1.0	7.2	7.1	3.5	9.2	23.1	6.3	30.0	1.6	10.4	31.3	28.7	72.3
Well															
8	9.6	5.4	3.8	7.8	62.7	56.8	84.7	360.8	29.5	140.8	2.5	48.6	426.1	252.7	730.2
9	9.3	7.5	1.8	7.6	19.2	15.9	46.4	67.2	14.0	107.6	1.2	23.1	99.4	171.1	294.8
4	7.2	6.0	2.9	7.3	13.1	14.2	8.7	19.8	19.5	93.9	0.6	15.4	25.9	119.7	163.7
11	6.0	6.1	1.8	7.9	23.0	3.2	11.2	17.4	13.5	87.9	0.0	22.3	22.9	93.1	140.5
2	5.1	5.3	1.1	7.4	18.3	3.7	8.8	12.2	12.6	78.0	1.9	20.7	15.3	77.7	115.6

Perusal of the examples given indicates that the loss of weight in either surface or well waters is not in proportion to the amount of dissolved oxygen or carbon dioxide, when these gases are present in limited quantities. Assuming that dissolved oxygen and free carbon dioxide indicate the degree of corrosiveness of the water, we would expect that the greater the amount of calcium, magnesium and sodium ions the less the degree of corrosion. If the anions are partly responsible for the corrosive properties of water, an increase of anions should result in an increase in corrosion. Taking the same examples of natural waters again we find (table 3) that with surface waters there is a tendency to become less corrosive when the cations increase, but definitely the reverse in well waters. The same holds for the total anions.

TABLE 3

Relations between total cations and total anions and corrosion

NUMBER		LOSS OF WEIGHT	TOTAL CATIONS	TOTAL ANIONS
Surface	10	8.4	10.8	34.5
	1	7.9	26.2	71.3
	7	7.4	39.4	107.3
	3	6.3	19.8	59.4
Well	8	9.6	204.2	531.1
	9	9.3	81.5	188.8
	4	7.2	36.0	133.2
	11	6.0	37.4	118.8
	2	5.1	30.8	102.8

Several of the examples given contained practically the same amount of oxygen but increasing quantities of carbon dioxide and anions as shown in table 4. These results seem to indicate that with the same amount of oxygen present corrosion increases with increasing amounts of carbon dioxide, bicarbonates and total anions, when the sources of water are taken into consideration. This would mean that the increased rate of corrosion may be attributed to the free carbon dioxide alone or to a combination of carbon dioxide, bicarbonate and total anions.

Carbon Dioxide Corrosion

It is well known that carbon dioxide is corrosive in distilled and natural waters. Nevertheless, experiments were made to check the

results obtained in the various natural waters. The various experiments can be grouped as follows: (1) Effect of carbon dioxide in distilled water without dissolved oxygen; (2) Varying quantities of carbon dioxide in natural water without dissolved oxygen; (3) Varying quantities of carbon dioxide in the presence of definite amounts of dissolved oxygen in natural waters. Preparation of the various waters for experimentation was as follows:

1. Distilled water was boiled to drive out dissolved gases while placed in containers; iron wire submerged and carbon dioxide gas introduced. The quantities of carbon dioxide in the water varied from 1 to 28 p.p.m.

2. Two natural well waters were selected (corresponding to Nos. 2 and 11), boiled to remove dissolved gases and carbon dioxide introduced. The boiling reduced the bicarbonates and another set was

TABLE 4

NUMBER	LOSS OF WEIGHT	D.O.	CO_2	TOTAL ANIONS	$^-HCO_3$
Surface 7	7.4	6.1	2.5	107.3	65.9
3	6.3	6.0	1.0	59.4	30.0
Well 4	7.2	5.2	2.9	133.2	93.9
11	6.0	5.3	1.8	118.8	87.9
2	5.1	5.3	1.1	102.8	78.0

prepared whereby the gases were removed as far as possible by vacuum and carbon dioxide introduced. The quantities of carbon dioxide in the water varied from 0.5 to 14 p.p.m. The dissolved oxygen content was from 0.2 to 0.3 p.p.m.

3. To the same natural waters, containing about 5 p.p.m. oxygen, varying quantities of carbon dioxide gas were added. The quantities of carbon dioxide varied from 1.8 to 16 p.p.m. All solutions were kept at 20–21°C.

Results obtained after 8 days are graphically shown in fig. 3. The boiled natural waters are omitted. The distilled water containing various quantities of carbon dioxide corroded the wire rapidly with amounts of carbon dioxide up to about 4 p.p.m., but with more carbon dioxide present the corrosion was relatively less for each additional p.p.m. This was probably due to the iron rust coating which protected the metal surface. When the duplicate series was vigorously

shaken after 4 days standing to remove a part of the iron rust, corrosion continued, but at a less accelerated rate.

The natural water with various amounts of carbon dioxide added showed less corrosion for each p.p.m. carbon dioxide present than the distilled water, but the rate of corrosion remained constant to about 8 p.p.m. carbon dioxide. With higher amounts of carbon dioxide the corrosion effects were very similar to the unshaken distilled water samples. The samples of natural water containing 5.2 p.p.m. dissolved oxygen to which varying amounts of carbon dioxide

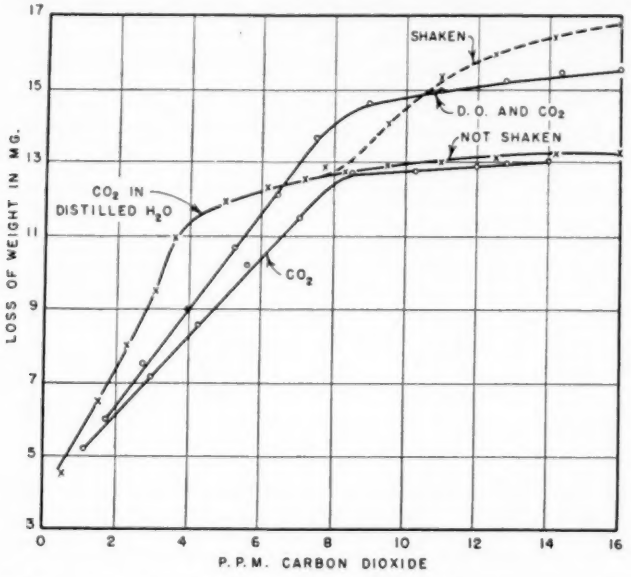

FIG. 3. Effect of Carbon Dioxide and Dissolved Oxygen on Corrosion in Natural and Distilled Waters

were added showed a higher degree of corrosion than in the absence of oxygen. Again corrosion decreased relatively when more than 8 p.p.m. carbon dioxide was present. Apparently the carbon dioxide affected the dissolved salts present, so that the corrosion products would be different in the natural waters as compared with the distilled water. The resistance of these salts to the transfer of oxygen would vary according to the type formed.

Taking the various results obtained with natural waters and plotting the 8-hour (or initial) corrosion against the dissolved oxygen (fig. 4) it appears that the initial corrosion may be the result of the

dissolved oxygen alone. When the 8-day corrosion is plotted for the various samples the results do not conform with the probable corrosion curve caused by oxygen, but are widely scattered. In order that the corrosion rate of iron in water be directly proportional to the oxygen concentration, it appears that the specific resistance of the corrosion product or the film produced should not change with varying oxygen concentrations. It is well known that it is possible to obtain corrosion product conditions which vary their resistance to the transfer of oxygen. Cox and Roetheli (2) showed that at low oxygen concentrations the corrosion products formed consisted largely of a granular black magnetic oxide, which was not resistant

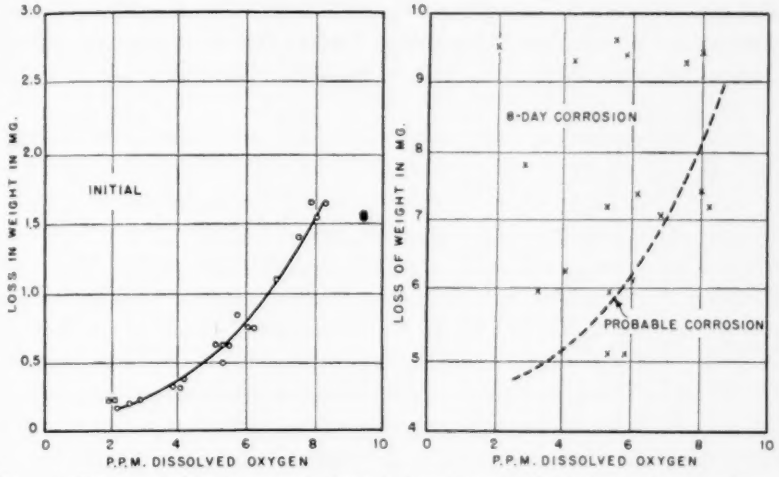

FIG. 4. Effect of Oxygen on Initial (8-hour) and Final (8-day) Corrosion.

to the transfer of oxygen, while at higher oxygen concentrations the protective gelatinous hydroxide was formed. The results given in fig. 4 show a nice correlation between the dissolved oxygen and the initial corrosion rate, but after a film has been built up the corrosion rate may increase or decrease with the resistance of the corrosion products to transfer oxygen.

In regard to the resistance of corrosion products to the transfer of oxygen those products which are dense and adhere closely to the surface give the most protection. These products are formed at the surface either by oxidation of the iron to ferric hydroxide, or by the salts present in the natural water. The granular magnetic products

and those products formed some distance away from the corroding surface have no value for protection. The existence and formation of the protective coating is affected by many factors. The alkali film of calcium and magnesium salts are affected by the free carbon dioxide in the water. In the presence of carbon dioxide such films will partially dissolve into solution as bicarbonates. The ferrous bicarbonates formed remain in solution as long as the chemical equilibrium is balanced, but ferrous bicarbonate is not a stable compound. In the presence of oxygen, ferrous bicarbonate is oxidized to ferric hydroxide:

$$3\ Fe(HCO_3)_2 + O_2 + 2H_2O = 4\ Fe\ (OH)_3 + 8\ CO_2$$

The carbon dioxide is reproduced. The importance of carbon dioxide appears to be not an independent factor, but a secondary factor depending upon its equilibrium in relation to the alkalinity of the water.

With respect to the corrosiveness or aggressiveness of the carbon dioxide present there are three possibilities: (1) The carbon dioxide concentration may be exactly that which the equilibrium demands, (2) the carbon dioxide concentration may be greater than the equilibrium demands, and (3) the carbon dioxide concentration may be less than the equilibrium demands. In the first and third cases the carbon dioxide may not be corrosive or aggressive, unless a source of carbon dioxide is made available by decomposition of bicarbonates, but in the second case the carbon dioxide is undoubtedly corrosive. In the presence of oxygen and aggressive carbon dioxide we may expect more corrosion than from aggressive carbon dioxide alone (fig. 3).

The relationship between carbon dioxide and bicarbonates has been given in the literature for several artificial waters. Tillman gives a table for free carbon dioxide and bicarbonate concentrations which are in equilibrium, together with the corresponding pH values. This gives the theoretical equilibria, but in natural waters such high carbon dioxide concentrations are rarely, if ever, found, and the salts concentrations of natural waters vary.

Plotting the loss of weight after 8 days against the total bicarbonates present in the natural well and surface waters, irrespective of the total dissolved oxygen and free carbon dioxide, we find some interesting results (fig. 5). In well waters corrosion increased directly with the total bicarbonates up to about 100 p.p.m. while with higher

bicarbonate concentrations the corrosion remained rather constant. In the case of the relatively soft surface waters the tendency appears to be in the opposite direction. This may be due to the effect of other factors influencing corrosion, overbalancing the effect of the bicarbonates. It should be remembered that these natural waters come from different sources and are influenced by the geological formations.

When the total anions were plotted against the loss of weight, the relationship was similar as compared with the bicarbonates, but the curve was more nearly a straight line up to 175 p.p.m. total anions. With the surface waters again the corrosiveness appeared to decrease with an increase in total anions.

Since there seemed to be a rather close correlation between total bicarbonates and the loss of weight the CO_2- $^-HCO_3$ ratio plotted

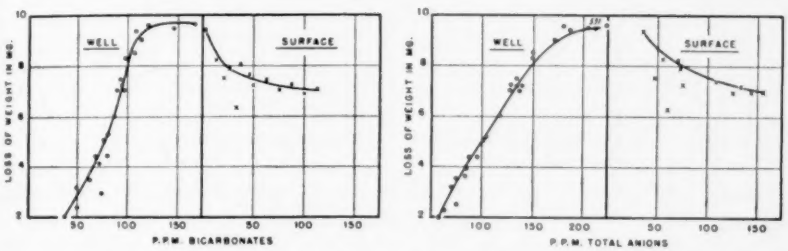

FIG. 5. Relation between Corrosion and Bicarbonates in Well and Surface Waters

against loss of weight should give an even better correlation. This was not the case; the points were more scattered. However, in these instances the dissolved oxygen present was left out of consideration. The dissolved oxygen would be a disturbing factor in such relationships, because it behaves differently under different conditions and varying salt concentrations. When the dissolved oxygen is kept constant and the loss of weight plotted against the CO_2- $^-HCO_3$ ratios practically straight line relationships result (fig. 6), except for those waters containing extraordinary low and very high salt concentrations.

The fact that the straight line relationships are the same in character for lower and higher initial dissolved oxygen concentrations, but appear to be at a lower level when the dissolved oxygen is lower, indicates that (1) the amount of dissolved oxygen in a given CO_2- $^-HCO_3$ concentration is the primary factor for corrosion, and (2) the

CO_2- $^-HCO_3$ concentration determines the degree of secondary corrosion. The results show also that the same or greater corrosion may result from low oxygen concentrations and high CO_2- $^-HCO_3$ ratios as from high oxygen concentrations and low CO_2- $^-HCO_3$ ratios.

Of more than passing interest is the fact that the behavior of surface waters is similar to that of well waters when results are presented on the basis of corrosion caused by the primary and secondary factors. These results indicate that the apparent different behavior of surface waters was caused by other, probably so-called tertiary, factors. These factors appear to be the amount and type of cations

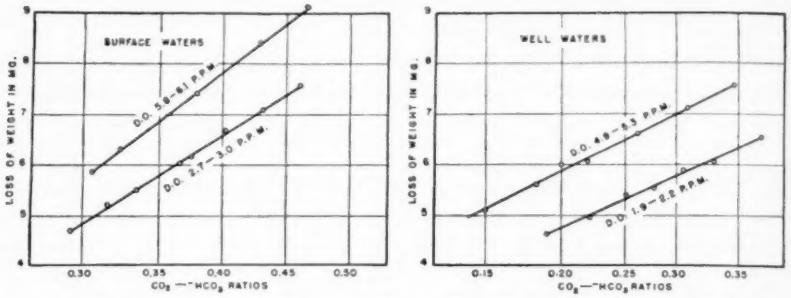

FIG. 6. Relation between CO_2-$^-HCO_3$ Ratios and Corrosiveness with Constant Oxygen Concentrations

preventing secondary corrosion and the concentrations of sulfates and chlorides.

Sulfates

The total anion-cation ratios of the different natural waters varied within narrow limits, namely between 2.32 and 3.33, and could not be directly correlated with the corrosiveness of the waters. When the ratios between the cations (Ca, Mg, Na and K) and the total bicarbonates were determined, curves could be constructed for the surface and well waters which indicated again that the bicarbonates were an important factor in the corrosion produced after 8 days, but that probably other anions than the bicarbonates played a minor role, causing the corrosion curve to deviate from a straight line. This is illustrated in fig. 7, where the results of a number of well waters are graphically shown.

Studies made on the effect of sodium sulfates on the corrosiveness of distilled water containing from 10 to 400 p.p.m. sulfates, showed

a practically constant loss of weight, but no increasing or decreasing loss of weight with increasing quantities of sulfates. The results show, however, a remarkable decrease of iron in solution with increasing amounts of sodium sulfate. Sodium chloride showed corrosion (and sodium carbonate to a less extent) but did not show the same decrease of iron in solution when amounts ordinarily found in natural

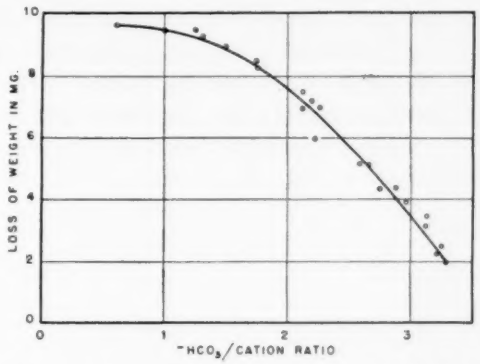

FIG. 7. Relation between Corrosiveness and $^{-}HCO_3$-Cation Ratios

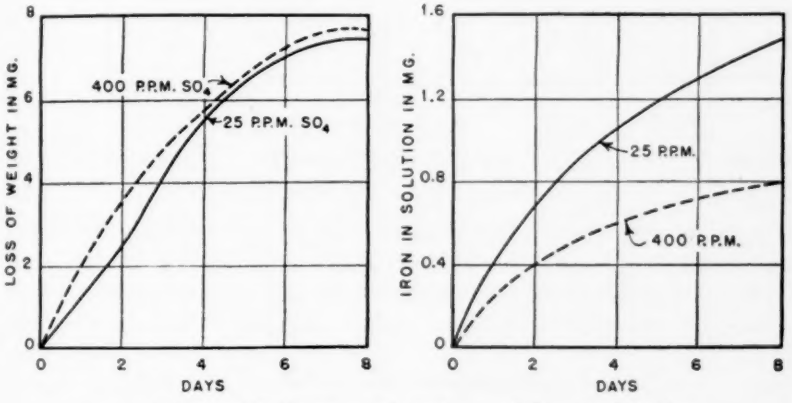

FIG. 8. Effect of Sulfates on Corrosion and Iron in Solution

waters were used. The sulfates formed a rather effective protective coating on the surface of the iron. As examples the results of experiments using 25 and 400 p.p.m. sodium sulfate are given in fig. 8. Although the rate of corrosion and the final total corrosion are about the same for the two salt concentrations, the iron in solution is materially less with the higher salt concentration.

The effect is apparently not in direct proportion to the amount of sulfate present, because when the results are plotted an S type curve results, similar to the shape of a growth curve (fig. 9). The growth of the rust on the surface of the iron reduces gradually the amount of iron in solution. Under ordinary conditions in natural waters some of the iron would be brought in solution. In the ranges below 100 p.p.m. and above 300 p.p.m. sulfates the effects of sulfates present would be most noticeable. Most of the water samples analysed and tested contained less than 100 p.p.m. sulfates, although some fell between 100 and 300, and some over 300 p.p.m. The analyses of selected samples given in table 1 are good examples. Since ferric sulfate is a more stable compound than ferric carbonate or ferric bicarbonate, the formation of a sulfate coating would be more effec-

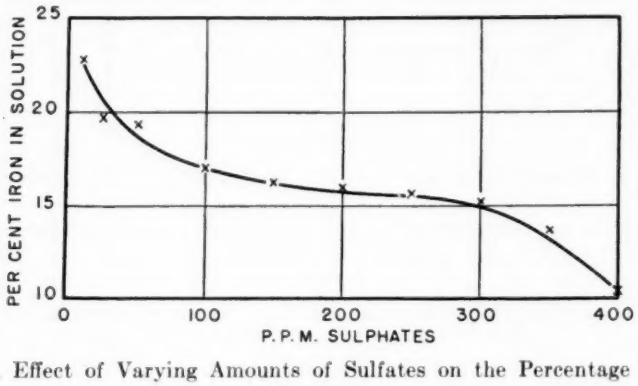

FIG. 9. Effect of Varying Amounts of Sulfates on the Percentage Iron in Solutions

tive in retarding secondary corrosion than the carbonate or bicarbonate coatings, aside from the fact that the sulfate and bicarbonate radicals produce secondary corrosion of different intensity. The corrosion caused by sulfates would be considered a tertiary factor.

Mechanism of Corrosion

Several of the theories and hypotheses advanced on the mechanism of corrosion have a tendency to overemphasize one or another particular factor involved. It is not the intention to propound a new hypothesis, nor to discuss those published. The results presented thus far indicate that more must be learned about the secondary and tertiary factors in conjunction with the primary causes of corrosion

before we will be able to solve the various problems. It is possible that in natural waters the initial corrosion can be designated as an electro-chemical process, so that the initial corrosion rate is determined by the dissolved oxygen present. The initial corrosion rate may be accelerated by the free carbon dioxide present. The total corrosion is undoubtedly affected by the free carbon dioxide and bicarbonates. If the initial corrosion is due solely to oxygen, the ionic iron will unite with the bicarbonate to form ferrous bicarbonate. This ferrous bicarbonate remains in solution if no other salts are present. In the presence of sulfates and chlorides a more stable ferrous sulfate or ferrous chloride may be formed and be further oxidized to ferric sulfate and ferric chloride, or the $Fe(HCO_3)_2$ may be directly oxidized to ferric hydroxide. These ferric compounds may or may not have protective value, depending upon whether they are formed at the surface or some distance away from the surface. In the first case they will closely adhere to the surface and provide protection. In the second case the product will precipitate out as a rust. The various possibilities undoubtedly depend upon the chemical equilibrium of the carbon dioxide, bicarbonates, sulfates and chlorides. Since the equilibria vary in different natural waters, the secondary corrosion factors may or may not play an important rôle in initial and total corrosion.

The formation of a protective film will retard or inhibit the oxygen approach to the metallic surface and thus retard or inhibit the electrochemical corrosion process. After the formation of a film the corrosion rate would be dependent upon the protectiveness (dense, porous, closely adhering, etc.) of the film, but the protective film itself depends upon the chemical equilibria of the various salts and gases. It is clear, therefore, that considerably more work is necessary before the various factors involved in the corrosion process can be properly evaluated. This must be done before theories which will adequately cover all variants of corrosion can be formulated. This is well illustrated by the findings that natural waters with the same dissolved oxygen content increase in corrosiveness when the dissolved salt constituents increase, but not at a proportionate rate and varying with the type of the constituents.

It is the hope of the writers that this work will stimulate further study of the secondary and tertiary corrosion factors under varying conditions and with different types of natural waters.

Summary

A number of natural surface and well waters were analyzed and their corrosiveness determined by different methods. An attempt was made to correlate the chemical constituents of the waters with the degree and rate of corrosion. The chemical analyses presented are examples and the terms "more corrosive" or "less corrosive" are used in a relative sense.

The data obtained could not be definitely correlated with the chemical constituents and the degree of corrosiveness of each individual natural water, but they give definite indications that primary, secondary and tertiary chemical factors play a rôle in the initial corrosion, and the rate and degree of corrosion. Such factors are exemplified by oxygen, carbon dioxide, bicarbonates, sulfates and chlorides. Under certain conditions secondary factors, such as carbon dioxide and bicarbonates, may be as important or more important than the primary factor oxygen. The chemical equilibria of dissolved salts and gases should be given important consideration in the study of corrosion phenomena.

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Discussion by *Samuel Shenker.* * Dr. Rudolfs and Mr. Wong have presented a laboratory attack upon the problem of the *corrosion of iron* which classifies the factors affecting this phenomenon. According to my understanding of this work, initial corrosion is due to the primary factor oxygen, with acceleration of the rate of this initial corrosion with increased concentrations of oxygen (fig. 4); then if carbon dioxide is present also, this factor is about as important as the oxygen factor. The bicarbonate and other anions are secondary factors, and there are still others such as cations which are tertiary in nature. In short, this work confirms earlier work along these lines by John R. Baylis and others who found that corrosion is not a simple phenomenon that can be explained by a single chemical

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reaction, but is complex and requires careful analysis during its inception and progression. This paper is commendable insofar as it calls our attention to the relative importance of the factors in initial and total corrosion of iron as here used.

In fig. 4, if we should project the curve towards the origin, it is easy to see that there would be no loss of weight of iron at zero, or even a few tenths parts per million dissolved oxygen. This, of course, has already been demonstrated on a plant scale by Shepard Powell's vacuum deaeration method. The matter of the progression of corrosion being due to dissolved oxygen has been known for some time. While oxygen continues to be a factor after the onset of corrosion, the progress of corrosion is affected by secondary factors such as the CO_2- $^{-}HCO_3$ ratio, bicarbonate and other anions such as sulfates and chlorides, and also the cations. This is illustrated in the total corrosion curve (fig. 4) in which it is seen that the points above the initial corrosion due to oxygen, are scattered and only a few are close to the "probable corrosion" curve as compared to the excellent correlation in the initial corrosion curve.

There is an apparent contradiction in fig. 5 since the well waters cause increased loss of weight of iron with increase of total anions and total bicarbonate, which is what one would expect, while the opposite is true in the case of surface waters. Probably, there is a factor that has not been determined in this case which would explain this condition. The corrosivity also is found to increase with increase in concentration of dissolved solids.

We have made great progress in the control of corrosion and studies such as these should stimulate our interest toward obtaining optimum conditions for minimum corrosion, thus effecting economy in numerous ways in the water works field.

New Knowledge of Fluorine in Relation to Dental Caries

By Gerald J. Cox

A CHARACTERISTIC of all methods which have been offered for the prevention or amelioration of dental caries is that they depend upon individual initiative. Such a condition means that no sound method could be wholly successful, men being what they are. But recent evidence, coming concurrently from three research groups and from three distinct approaches, suggests a means of very materially reducing the incidence of dental caries by a procedure that is applicable to whole communities. Furthermore, the prophylaxis could be applied in such a way that the individual would be hard put to escape the treatment.

In 1931 it was discovered (4, 12) that fluorine in water supplies is the causative agent in endemic mottled enamel. A mass of corroborative evidence has appeared and consequently water supplies have been changed or treated to protect children from dental fluorosis. That the present trend toward complete removal of fluorine from water and food may need some reversal is shown below.

It has been demonstrated recently by Dean (7) and by Dean, Jay, Arnold, McClure, and Elvove (8) that the incidence of dental caries in children is related to the water supply. The element most probably responsible for improved caries-resistance is fluorine; but because of other differences in the mineral composition of the waters, these authors could not state without reservation that increased fluoride content of a water accounted for the difference in caries incidence.

Cox, Matuschak, Dixon, Dodds, and Walker (5) produced, in rats, teeth relatively resistant to dental caries by means of sodium fluoride given during the period of formation of the teeth. This experimental

A paper presented on September 20, 1939 at the Western Pennsylvania Section Meeting at Johnstown, Pa., by Gerald J. Cox, Senior Fellow, Nutrition Fellowship of The Buhl Foundation, Mellon Institute, Pittsburgh.

procedure, in which *sodium fluoride was the sole difference* in the treatment of two groups of rats, indicates that the better teeth observed by Dean (7) and by Dean, Jay, Arnold, McClure and Elvove (8) is to be ascribed to the fluoride content of the water.

Armstrong and Brekhus (1) found that the enamel from sound teeth had an average of 110 p.p.m. of fluorine; the enamel of carious teeth *from the same mouths* had 69 p.p.m. This analytical result is additional evidence that fluorine acts to reduce dental caries.

Before it was known (4, 12) that fluoride in drinking water caused mottled enamel, it had been observed that dental caries was not increased by this condition of defective enamel. Black, with the collaboration of McKay (2), remarked on this lack of increased caries in mottled enamel. Bunting, Crowley, Hard, and Keller (3) found caries restricted to small pit and fissure lesions in a community with mottled enamel. McKay (9) used the failure of dental caries to develop extensively in teeth with mottled enamel as testimony that poor enamel structure is not conducive to dental caries.

We have pointed out (5), in consequence of the demonstrations of the relation of fluorine to dental caries, that much research on the cause of tooth decay must be re-evaluated. All dietary studies must be scrutinized to determine whether fluorine of the rations has played a part in the results. Dean, Jay, Arnold, McClure, and Elvove (8) have stressed the similarity of the climatic and social conditions of the Illinois cities which they surveyed, namely, Galesburg, Macomb, Monmouth, and Quincy. They presented data to show that there was no significant difference in sunlight among the communities. They did not cite evidence of variation of dietary habits. But, of course, marked differences in food usage would not be expected. One would suppose, for example, that the per capita consumption of fermentable carbohydrates would be about the same for each of the four cities.

As a result of the appearance of fluorine in a beneficent rôle in the prevention of dental caries, it is likely that theories involving the formation of caries-resistant teeth will receive much more approval in the future.

Fluoridization of Water

The addition of fluorine to the dietary in optimum amounts during the period of formation of teeth is obviously indicated (5).

It is suggested by Dean and his coworkers that teeth can be free of both mottled enamel and dental caries. But the threshold for the

appearance of mottled enamel, variously estimated at from 0.9 to 2 p.p.m., indicates that the beneficial range of fluorine content is very small. With the threat of the Scylla and Charybdis of dental caries and mottled enamel, great caution must be observed in the means of administration of fluorides and in the control of such procedures as may be adopted.

Treatment of water supplies, by the addition of fluorides in most cases, offers the most practical means of approaching the goal of sound teeth for all children—truly a mass prevention of dental caries. But no set level of fluoride content can be assigned for all waters. The objective is the optimum total amount of fluorine ingested per child from all sources.

As there are seasonal and climatic variations of water consumption, these factors must be studied for regulation of the fluoride content of the community water supply. For instance, during the winter a higher content of fluorine would be provided than during the summer. Dry countries would use a water of lower fluoride content than humid areas. For general planning and regulation of the fluorine level, a relationship between temperature and humidity and the water consumption of normal children would doubtless be of value.

It seems inescapable that, if water is used as the vehicle for fluoride, all the water of a community must be treated. Therefore, means to treat and to control must be considered. Addition of soluble fluorides, such as hydrofluoric acid or its alkali salts, would constitute a relatively simple treatment; use of the "insoluble" fluorides, such as fluorite or cryolite, would require preparation of large amounts of saturated water by some mechanism, such as a granulated mineral bed, with subsequent dilution.

Other Avenues

Prophylaxis by means of bottled waters, medicinals or special foods would not apply satisfactorily to the mass of the people because of general human negligence in such matters. Milk might be used as a medium but the greater variation in its consumption and the dangers of lack of control of treatment would be against such a procedure. Such means, however, must be examined as stop gaps pending treatment of community water supplies. Before fluoridization of water can begin there are other phases that must be examined. Fluorides are among the most toxic of substances. Mottled enamel results from as little as 0.0001 per cent of fluorine in drinking water. The results on adults of drinking water containing sufficient fluoride

to prevent dental caries in children must be determined. It may be predicted that women with body supplies of fluorine may transmit caries-resistance to their children both placentally and by lactation but no factual evidence is at hand. Miller (10) has shown that heavy feeding of calcium fluoride or sodium fluoride inhibits the formation of dental caries in *pre-formed* rat molars, but there is no proof that the low level of fluorine beneficial in the formation of teeth would be effective in the arrest of decay.

Except for protection of teeth from dental caries, no other need for fluorine has ever been shown. However, studies on fluorine have practically all been made on fluorosis; the consequences of afluorosis, if any, are unknown. Sharpless and McCollum (11) in the only published study of afluorosis found no apparent injury to rats raised on low fluorine diets. Further studies of this nature are urgently needed.

Cox, Matuschak, Dixon, and Walker (6) have recently produced mottled enamel in the *molars* of rats. Thus a means is provided for the experimental study of the relations of mottled enamel and dental caries in a single species. Dean, Jay, Arnold, McClure, and Elvove (8) have opened an area for field studies that is of vast importance for the clarification of the knowledge of the causal factors in dental caries. For example, the proponents of the oral environment theory of dental caries, particularly those who believe that fermentable carbohydrates *cause* as well as *promote* tooth decay, can readily test their theory by a survey of carbohydrate consumption in Galesburg and Quincy. Perhaps this "disease of civilization" can be eliminated by the observations on civilized subjects rather than by studies of small, restricted uncivilized groups.

Use of water to which a part per million of fluorine has been added for other than drinking can conceivably lead to undesirable effects. Plant and animal life may be injured. There may be interference with industrial uses. Every use of water must be examined as well as current water treatment practices before fluoridization can be begun.

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EDITOR'S NOTE: Abstracts of references 1 and 8 appear in this issue of the JOURNAL under the heading "Fluorides in Water." An abstract of reference 7 appeared in Jour. A. W. W. A., **31**: 1081 (1939).

Discussion by *Harry E. Jordan.** Shall city water supplies be specially treated to prevent decay of teeth? That is the latest issue raised by certain investigators who conclude that in some cities the prevalence of dental caries (or tooth decay) is associated with lack of fluorine in the public water supply. Water works men have heard a great deal about too much fluorine in city supplies, but it is now suggested that certain other cities drawing their water supplies from surface sources and preparing them for use by efficient modern purification methods, are furnishing a supply that contains too little fluorine.

Is it advisable to counteract this by adding fluorine to the water? Several years ago water works men were advised by some investigators that they could help reduce goiter in certain cities by adding iodine. A few cities tried it, only to learn that the medical profession felt that greater benefits would be attained by direct treatment for those who clearly suffered from iodine deficiency.

Between mottled enamel from too much fluorine and dental caries from too little, lies a field for much study by dietitians, physiological chemists, dentists and doctors so as to discover just how the diet and drink intake of the individual contributes to the making and maintenance of good tooth structure.

Water works engineers need the evidence of wide research along with dental and medical approval before they are led into mass medication through public water supplies. Their task is the production of safe and palatable water supplies, adequate in amount constantly served at reasonable cost. They must await widespread professional support before expanding the scope of their activities.

*Secretary, A. W. W. A.

A Uniform Basis for Activated Carbon Comparison

By W. A. Helbig

IN A recent paper[1] discussing the relative merits of the phenol value test and the threshold odor test as procedures for evaluating activated carbons for water treatment, it was pointed out that neither of these tests as it is constituted at present affords a basis of carbon comparison whereby adsorption tests on one type of odor can be correlated with tests on another type.

This paper is a progress report on work that has been done during the past twelve months, in an effort to find a common basis of comparison. The goal sought is a basis whereby odor removal tests at any water works plant can be compared with similar tests at any other plant, and whereby a laboratory evaluation of a carbon will rate it correctly with respect to odor adsorptive capacity on any type of odor. This goal has not yet been reached, but considerable progress has been made.

In previous reports[1, 2] presented before this Committee, it was definitely established that adsorption of odors with powdered activated carbon follows the Freundlich adsorption equation, when odor intensity is expressed in threshold odor units.

By means of adsorption isotherms plotted in accordance with this equation, a series of carbon samples can be compared for relative odor adsorptive capacity, on a particular sample of odor bearing water, containing a particular type of odor. The comparison is based on the carbon dosages required in each case to reach the same odor residual (fig. 1). Because the adsorption isotherms obtained with a

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A report presented on June 12, 1939, before the Committee on Activated Carbon Research at the Atlantic City Convention, by W. A. Helbig, Chemical Engineer, Darco Corp., New York, N. Y.

[1] Comparison of Tests for Activated Carbons, Jour. A. W. W. A., **30:** 1225 (1938).

[2] Taking the Guesswork Out of Activated Carbon Dosage, Jour. A. W. W. A. **30:** 320 (1938).

series of carbon samples are in general not parallel, the relative efficiencies of the carbons will vary with the odor residual at which the efficiencies are measured.

The primary requisite, therefore, for a common basis of comparison of carbons is that their adsorptive capacities be compared in all cases at the same residual concentration of adsorbate.

At first glance, it may appear that this condition is satisfied if carbons are always compared at the same arbitrary residual odor, and

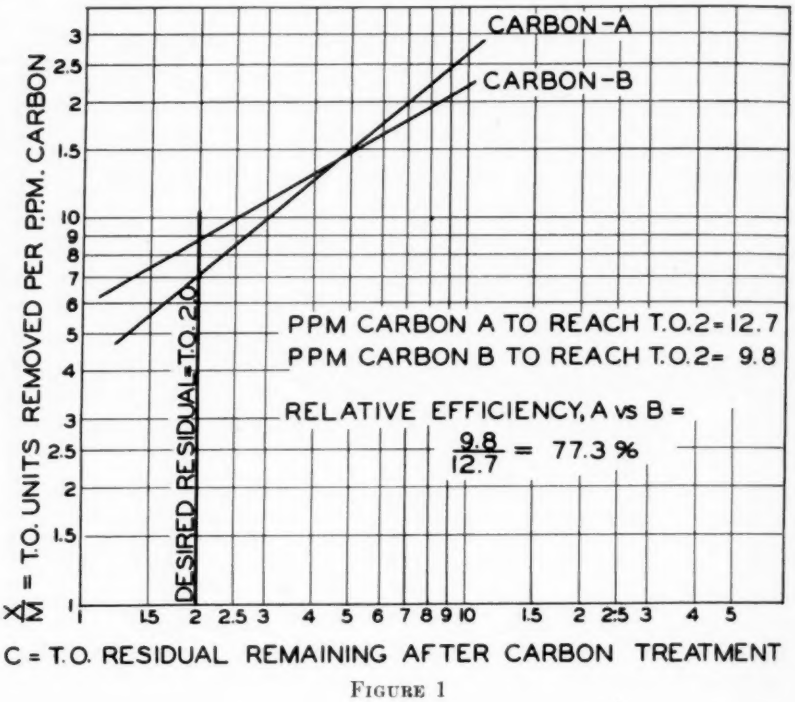

CARBON-A
CARBON-B
PPM CARBON A TO REACH T.O.2=12.7
PPM CARBON B TO REACH T.O.2= 9.8
RELATIVE EFFICIENCY, A vs B =
9.8/12.7 = 77.3 %
DESIRED RESIDUAL=T.O. 2.0
X/M = T.O. UNITS REMOVED PER P.P.M. CARBON
C = T.O. RESIDUAL REMAINING AFTER CARBON TREATMENT

FIGURE 1

that on this basis relative efficiencies obtained with a set of carbons at one plant may be expected to be the same as the efficiencies obtained with these carbons at another plant, provided that the same procedure was used at both plants.

This is not the case, however, and to understand the reason for this, it is necessary to examine somewhat more closely just what is involved in making threshold odor measurements, and what these measurements mean in terms of actual concentration of odor-causing substance.

The investigation of these points constitutes the first phase of the work reported in this paper. Briefly summarized, the data obtained indicate that the same odor intensity, expressed in threshold odor units, may involve vastly different actual concentrations of odor-causing compounds.

In other words, unless the same odor-causing compound is present in all cases, the use of an arbitrary threshold odor residual as the common basis for adsorption isotherm comparison between different plants is invalid, because the comparisons would not be based on the same residual concentration of adsorbate.

It is realized of course that it is practically hopeless to expect to be able even to identify the substances which cause odors in water supplies, and still more hopeless to determine their actual concentration.

However, it is not hopeless to ascertain, with odor-causing compounds of known composition and purity, a general relationship between odor intensity, actual concentration, and adsorption on carbon. Having established the correct basis for obtaining the same relative efficiencies with a series of carbon samples on solutions of a number of such compounds, the principle can be extended to comparisons of carbons on odors of unknown composition and concentration.

For work along this line, the following compounds were investigated: phenol, p-chlorphenol, coumarin, vanillin, heliotropin, safrole, methyl salicylate, and terpineol.

These compounds were all tested for consistency of odor intensity in proportion to concentration, it being anticipated that in the very dilute solutions used, some of them, because of hydrolysis, oxidation, or other causes, might not give a perfectly constant relationship between odor intensity and gravimetric concentration over the range of concentrations employed.

The test consisted in preparing solutions of known gravimetric concentration, and determining the odor intensity for each concentration by means of the threshold odor test, the samples being smelled at 65°C. in 500 c.c. Erlenmeyer flasks, without the use of a nose-piece.

The results obtained with phenol are given in table 1, and graphically represented in fig. 2, which shows the straight line relationship between gravimetric concentration of phenol and odor intensity at various concentrations.

TABLE 1

Threshold Odor Value of Phenol Solutions at Various Concentrations

PHENOL CONC.	T.O. VALUE	G_p	DEVIATION FROM AVER. G_p	PER CENT DEVIATION
p.p.m.				
75	11.0	6.75	−0.38	5.65
150	20.8	7.21	+0.08	1.11
300	43.0	6.98	−0.15	2.15
450	65.8	6.84	−0.29	4.25
600	83.3	7.21	+0.08	1.11
750	104.2	7.20	+0.07	0.97
150	22.7	6.61	−0.52	7.85
450	65.8	6.84	−0.29	4.25
600	83.3	7.21	+0.08	1.11
750	104.2	7.20	+0.07	0.97
825	113.6	7.27	+0.14	1.93
150	21.7	6.91	−0.22	3.18
225	31.2	7.21	+0.08	1.11
375	50.0	7.50	+0.37	4.94
675	92.6	7.34	+0.21	2.86
900	119.0	7.57	+0.44	5.81
525	71.4	7.35	+0.21	2.85
540	85	6.35	−0.78	12.3
360	50	7.20	+0.07	0.97
360	55	6.55	−0.58	8.8
720	100	7.20	+0.07	0.97
540	75	7.20	+0.07	0.97
360	50	7.20	+0.07	0.97
180	26	6.93	−0.20	2.89
180	26	6.93	−0.20	2.89
360	50	7.20	+0.07	0.97
180	25	7.20	+0.07	0.97
540	75	7.20	+0.07	0.97
720	100	7.20	+0.07	0.97
720	100	7.20	+0.07	0.97
360	55	6.55	−0.58	8.8
180	24	7.50	+0.37	4.94
540	75	7.20	+0.07	0.97
720	100	7.20	+0.07	0.97
360	50	7.20	+0.07	0.97
180	24	7.20	+0.07	0.97
540	75	7.20	+0.07	0.97
720	100	7.20	+0.07	0.97
Arithmetic Average.....		7.13	0.196	2.75

This graph illustrates an interesting and important relationship between odor intensity and concentration of odor-causing substance. It also helps to clarify the basic principle of the threshold odor test.

It will be seen that the threshold odor value increases in direct proportion to concentration of phenol over the entire range of concentrations tested. As the phenol concentration is doubled, the threshold odor value is doubled; as concentration is tripled, the value is tripled, etc. This straight line proportionality is evidence that phenol does not undergo chemical changes (oxidation, hydrolysis,

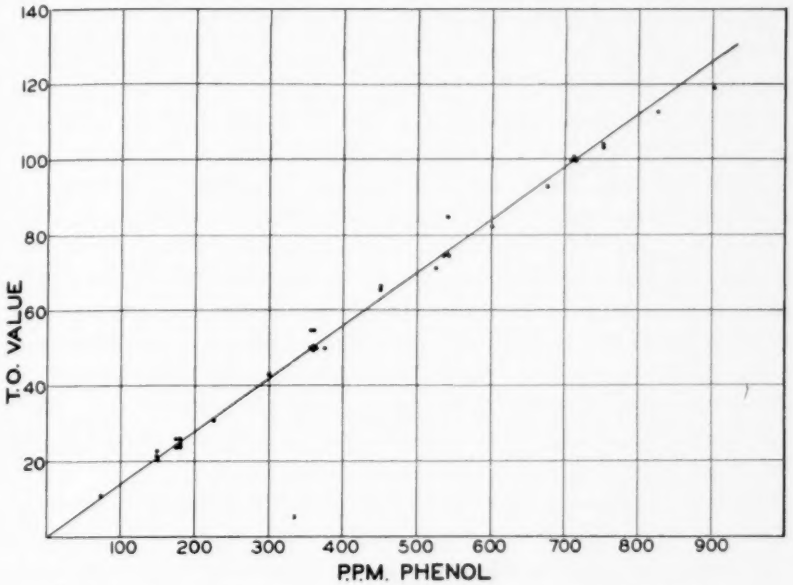


FIG. 2. Gravimetric Concentration of Phenol vs. Threshold Odor

polymerization) or physical changes (dissociation, association) as its concentration varies, which affect its odor intensity. On this score, therefore, phenol was considered a satisfactory test material for further work.

The next consideration was the facility with which the odor threshold could be found. In conducting the threshold odor test, it will be recalled, the sample of odor-bearing water is diluted to the point at which its odor is just barely perceptible, such that with further dilution the odor intensity falls below the threshold of perceptibility.

Now, in the scale of units employed, if a sample has an odor in-

tensity such that on dilution with a finite quantity of odor-free water the odor is no longer perceptible, the sample has a threshold odor value of one (T.O. 1).

It follows therefore that in any threshold odor determination, the odor intensity of the diluted sample at which the threshold is found, is T.O. 1.

In other words, the threshold odor test consists in finding the dilution necessary to reach T.O. 1. This means that in a series of tests involving a particular odor, whether of known or unknown composition, the same gravimetric concentration of odor causing substance is reached at the threshold of perceptibility.

When working with odor-causing compounds of known composition and concentration, it is possible to check the consistency with which the threshold point, T.O. 1, can be found, because this corresponds to a definite gravimetric concentration of the compound.

If, for example, the observer conducts a threshold odor test on a sample of phenol solution, having a concentration of 36 p.p.m., and reports an odor of 5, it means that at a dilution of 50 c.c. of sample to 250 c.c. with odor-free water (i.e., a five-fold dilution) he found the threshold of perceptibility. The concentration of phenol at this dilution was $\frac{1}{5}$ the original concentration of 36 p.p.m. or 7.2 p.p.m.

He then runs an odor test on another phenol solution, having for example a concentration of 85 p.p.m., and reports T.O. 12. This involved a twelvefold dilution, and a concentration of phenol at the threshold of perception of 85/12 or 7.1 p.p.m.

Calling the gravimetric concentration of the odor causing substance G, the odor at this concentration $T.O.$, and the gravimetric concentration at the threshold of odor perceptibility G_p, we can write

$$\frac{G}{T.O.} = G_p, \text{ a constant for the compound tested.}$$

This value G_p is shown in table 2 for each of the odor determinations run on phenol solutions of various concentrations. The per cent deviation of each G_p value from the average of all the values is also shown. It will be seen from this that the threshold point for phenol can be determined consistently with little experimental error. Phenol therefore met the second requirement for a satisfactory odor test material, viz. facility with which the threshold point can be found.

Similar tests were run with the other test compounds, the results

of which are summarized in table 2. The plots of gravimetric concentration vs. T.O. value are shown in figs. 3–9.

Table 2 indicates first of all the wide variability in concentration at the threshold of odor perceptibility, for different compounds, as shown by the G_p values listed. These range all the way from 7.13 p.p.m. for phenol, which appears to be exceptionally high, to 0.0349 p.p.m. for methyl salicylate. It should be evident now why an arbitrary T.O. value cannot be used as a common basis for comparing odor-adsorptive capacity of carbons on all types of odors, because such an arbitrary T.O. value will represent a different gravimetric

TABLE 2

Résumé of Threshold Odor Test Results with a Series of Odor-Causing Compounds

COMPOUND	NO. OF DETERM.	AVER. G_p VALUE	AVER. C_a ERROR IN G_p	BOILING POINT	MOL. WT.	MOLAR CONC. OF G_p $\times 10^{-7}$	VAPOR PRESSURE AT 65°C.	VAPOR PRESSURE AT G_p CONCENT. MM. $\times 10^{-9}$
		p.p.m.		°C.			*mm.*	
Phenol	41	7.13	2.75	182	94	758	7.4	10,120
Methyl salicylate	41	0.0349	7.23	223	152	2.29	1.9	7.84
p-chlorphenol	20	0.1204	7.18	217	128.5	9.36	2.3	38.8
Terpineol	35	0.1042	30.2	220	154		2.4	
	*29	0.0860	6.16			5.58		24.2
Heliotropin	11	0.0620	9.35	263	150	4.13	0.32	2.38
Vanillin	32	0.1533	22.1	285	152	10.10	0.09	1.64
Safrole	43	0.0407	18.0	234	162	2.51	0.65	2.94
Coumarin	24	0.1380	41.2	302	146	9.45	0.05	0.85

* Omitting six determinations of inconsistently very large error.

concentration of adsorbate for each odor type involved. Carbon comparison from adsorption isotherms is valid only if all comparisons are made at the same concentration of adsorbate.

Table 2 shows also that the consistency with which the threshold of perceptibility can be found with a reasonable degree of accuracy varies markedly with different compounds. This is reflected in the average per cent error (average per cent deviation from an average G_p value) in the table. With phenol and methyl salicylate, particularly, this error is consistently small; with terpineol, p-chlorphenol, and heliotropin, it lies within reasonable limits, although for reasons given below, these compounds are not considered quite as

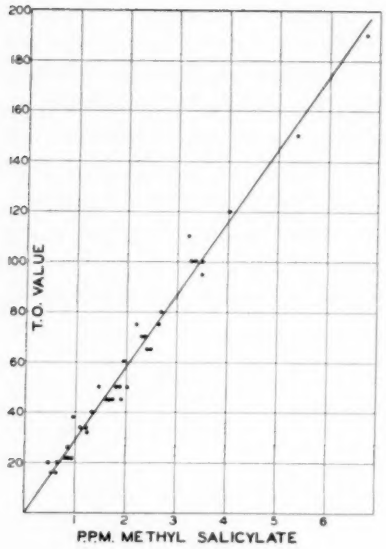


FIG. 3. Methyl Salicylate

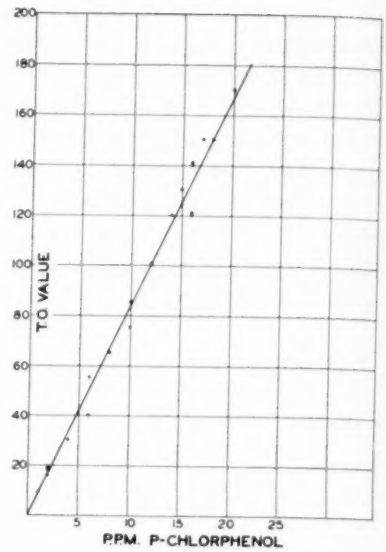


FIG. 4. P-Chlorphenol

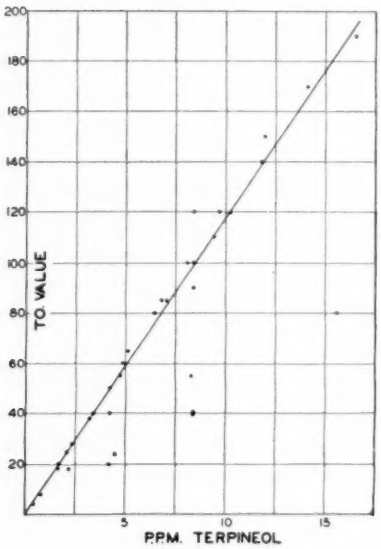


FIG. 5. Terpineol

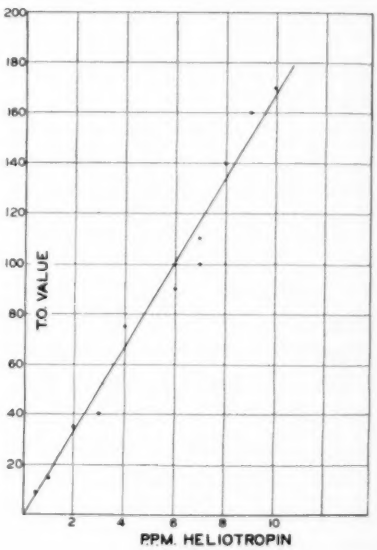


FIG. 6. Heliotropin

trustworthy as phenol and methyl salicylate. With safrole, vanillin, and coumarin, on the other hand, the errors are very large, and these compounds are therefore considered unsatisfactory for further work.

In considering the error involved with the various compounds tested, it is instructive to examine the graphical portrayal of their relationships between gravimetric concentration and threshold odor value.

As has already been pointed out, fig. 2, showing the phenol results, gives an excellent straight line plot with all the points falling close to the line. The same is true for methyl salicylate (fig. 3) and p-chlorphenol (fig. 4). In the case of p-chlorphenol, however, there are only 20 determinations, as against 41 for each of the first two compounds. Before p-chlorphenol is finally accepted, it is felt that it should show consistent results in further tests.

Terpineol (fig. 5) shows good results, with the exception of 6 points (out of 35) which have an inconsistently large error—so large, in the face of the consistently good other points, that their correctness is almost open to question. With these points eliminated, terpineol shows an error of only 6.16 per cent, and is therefore being retained tentatively for further work.

Heliotropin (fig. 6) shows fairly good results, but only 11 determinations were conducted with this compound, so that its reliability is still open to question. Should later stages of the investigation show the desirability of work with an additional compound, heliotropin will probably be subjected to more exhaustive tests.

Vanillin (fig. 7) shows an exceedingly interesting phenomenon. A study of this plot shows at once that the large G_p error listed for vanillin does not result from a scattering of the points; these fall very closely along the line drawn through them, but this line is not straight; it shows a distinct curvature. This is interpreted as an indication that vanillin undergoes some kind of change as its concentration increases, which change affects its odor-causing properties. Such a tendency makes it necessary to rule out vanillin as a dependable test material for odor absorption work.

About all that can be said for safrole (fig. 8) and coumarin (fig. 9) is that they are "bad actors." The odors of these compounds are very elusive; as the plots show very plainly, it is practically impossible to obtain consistent threshold odor values with them. Their very elusiveness, however, is interesting, because the writer has had a similar experience in field test work, on an odor occurring in the

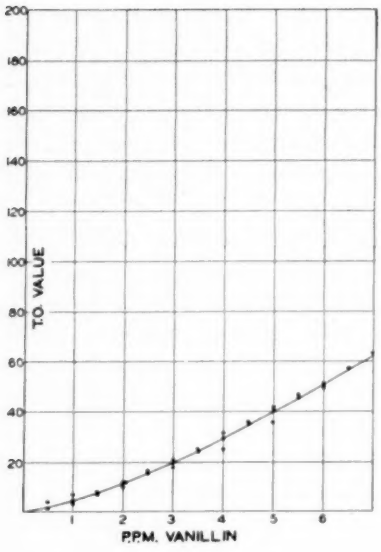


FIG. 7. Vanillin

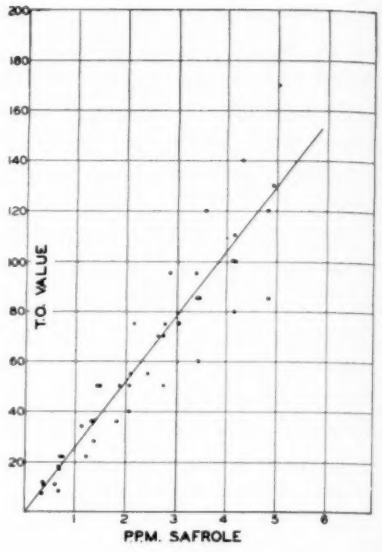


FIG. 8. Safrole

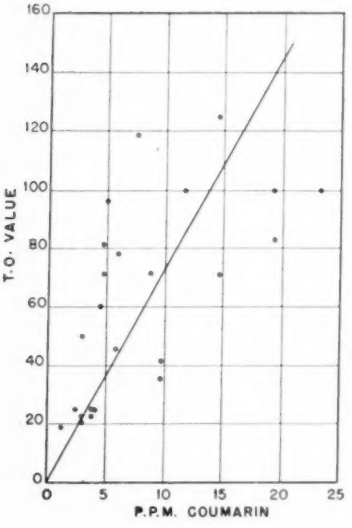


FIG. 9. Coumarin

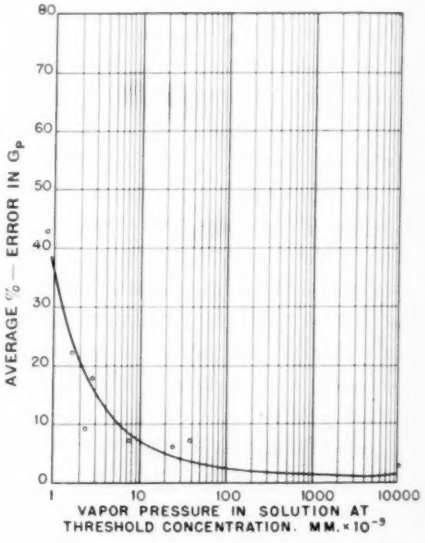


FIG. 10

FIG. 10. Apparent Expectancy of Error in Odor Determination in relation to vapor pressure of compound in solution at threshold concentration for 65°C. tests.

water at Cincinnati, Ohio, in which the threshold point was also tantalizingly impossible to check.

A study of the consistency with which the threshold of perceptibility can be found with the compounds tested, as indicated by the average G_p error, naturally led to a consideration of their physical properties. Since volatility is obviously important, and volatility is roughly indicated by boiling points, the errors in G_p were tentatively plotted against the respective boiling points, which are listed in table 2. A sufficient number of points fell closely enough to a curve drawn through them to arouse curiosity in investigating this relationship somewhat more closely.

The literature was therefore searched for vapor pressure data on the compounds tested, and sufficient data were found to warrant the application of Dühring's rule in calculating the vapor pressure of the pure compounds at 65°C., the smelling temperature. These figures are also listed in table 2. Their absolute values are possibly somewhat in error because of the necessity of making a small extrapolation in applying Dühring's rule, but their relative values are probably very nearly correct.

These vapor pressures were then corrected for the molar concentration of G_p (also listed) using Raoult's Law for reduction of vapor pressure, and the resultant figures are listed in the last column of table 2.

Figure 10 is a plot of these data (actually, relative volatility) against average G_p error. Considering the exceedingly large range of vapor pressures* involved—from 10, 120 $\times$ 10^{-9} mm. for phenol to 0.85 $\times$ 10^{-9} mm. for coumarin, the curve obtained seems to indicate a very definite relationship between volatility and accuracy of odor determination. This should be of considerable help in considering other odor-causing compounds for standardized threshold odor work.

Also, the concept of the G_p value is a powerful instrument in placing such odor work on a reproducible basis. By means of G_p determina-

* The reader is cautioned not to assume that the vapor pressure figures given represent the actual partial pressures of these compounds at their respective concentrations. Auxiliary calculations not here reported indicate that the solutions in question are not truly ideal solutions, which fact makes calculation of actual partial pressures impossible. The figures are given as "vapor pressures" only for want of a more suitable terminology, and are intended to convey nothing more than a reasonably close picture of relative volatility.

tions on suitable compounds, it is possible to ascertain quantitatively how closely different observers can agree in their ability to perceive odors. This constitutes an essential step in placing activated carbon comparison on a uniform basis. It is respectfully suggested, therefore, that a part of the Committee's program for the next year be the issuing of samples of pure compounds to committee members for odor tests along this line.

Another useful application of the G_p concept is in the comparison of odor determinations at different smelling temperatures. Considerable work has been done along this line by C. H. Spaulding of

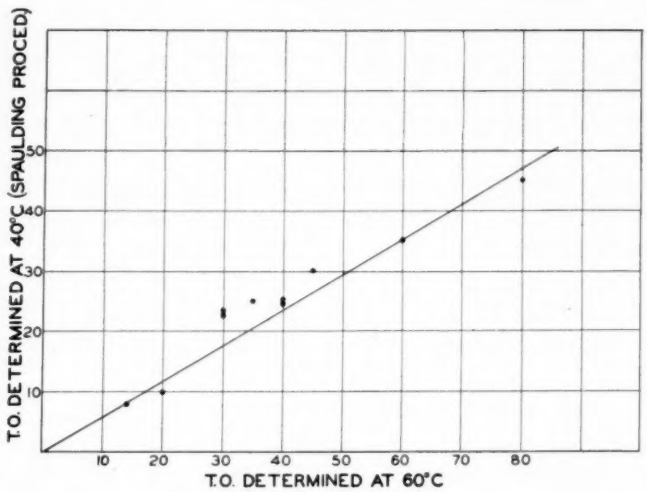


FIG. 11. Relation between T. O. Values Determined at 40° and 60°C. at Cleveland.

Springfield, Illinois, using 40°C. as the smelling temperature. The writer had an opportunity to observe this procedure in use some months ago at the Springfield laboratory, and even in this very brief experience it was very evident that the experimental conditions are much more favorable for the observer at a 40° smelling temperature than at 65°C.

Tests at 40°C., and also at temperatures between 40° and 65°, are under way at the Darco Experimental Laboratory in Wilmington, but enough results have not been obtained to date to warrant a report on them at this time.

However, using the G_p concept, certain facts become evident even

without experimental data. The original equation defining G_p is $G_p = \frac{G}{T.O.'}$, where, it will be recalled, G is the gravimetric concentration of the compound at which the odor intensity is $T.O.$ $T.O.$ is numerically equal to the dilution ratio at which the threshold was found, and G_p is therefore the gravimetric concentration at $T.O.$ 1, the threshold of perceptibility.

Now, this equation is completely defined only if the smelling temperature is specified, i.e. G_p is, of course, different at different smelling temperatures. For two different temperatures we can write.

$$\left[G_p' = \frac{G}{T.O.'}\right]_{T'}$$

$$\left[G_p'' = \frac{G}{T.O.''}\right]_{T''}$$

This is only another way of saying that for a particular concentration G, the threshold odor and therefore the G_p value, will be different at different temperatures.

Transposing the equations,

$$G = [G_p' \times T.O.']_{T'}$$

$$G = [G_p'' \times T.O.'']_{T''}$$

and combining, we have

$$G_p' \times T.O.' = G_p'' \times T.O.''$$

and finally

$$T.O.'' = T.O.' \times \frac{G_p'}{G_p''}$$

Since G_p' and G_p'' are both constants, this means that in general, odors determined at one temperature for a particular odor-causing compound have a constant relationship to the odors at another temperature, e.g., if on a series of samples the odor is determined at both 40°C. and 60°C. for each sample, the values obtained will plot against each other as a straight line.

Such a line was actually obtained from data secured by Messrs. Marsh and Irwin of the Baldwin Filtration Plant at Cleveland, who have very kindly permitted the writer to use the figures in this paper. The plot is shown in fig. 11. The slope of this line is the

ratio of the G_p values at 40° and 60°, although the absolute G_p values are not known, since these tests were run on regular samples of Cleveland water, in which the odor-causing substances were of course unknown. There is suggested in this the interesting possibility of classifying unknown odors in terms of their threshold odor value ratios at two fixed temperatures, and ascertaining their characteristics by comparison with a scale of similar ratios for known compounds. This may constitute one method of expressing odor character.

A more useful method of expressing odor character, however, is in terms of adsorbability on activated carbon. This will be one of the subjects to be covered in the second phase of this investigation. The primary object of further work will be to study the adsorption characteristics of the compounds established as reliable test materials in this report, with a series of different activated carbons. The end sought, as mentioned at the beginning of this paper, is a basis for comparing adsorptive capacities which will rate activated carbons at the same relative efficiencies regardless of the adsorbate used.

Standard Methods For The Examination of Water and Sewage

THE EIGHTH edition of "Standard Methods for the Examination of Water and Sewage" appeared in 1936. The first printing of 3000 copies was issued in June, 1936; the second printing of 3000 copies issued in March, 1937; and the third printing of 2000 copies issued in September, 1938. A fourth printing will issue late in 1939.

In a text of this character, errors are made by the editors or the printer as the material is prepared and such errors are corrected as found and as later printings issue.

The complete record of corrections made in the second and third printings, together with those indicated for correction in the forthcoming fourth printing follows. Each correction is numbered and there follows each number two, three or four asterisks (*) which indicate the printing in which the correction has become or will become effective.

Reference to the title page of the copy in the possession of any individual will disclose which printing he has. If, for example, he is using the second printing, correction numbers followed by two asterisks (**) have been made in that copy. Corrections marked with four asterisks (****) will not be made until the fourth printing issues late in 1939.

1.** Page 1. In *paragraph 3*, change "Parts III and VI" to read "Parts III and IV."

2.** Page 20. In *section 1.11*—Place quotation marks before and after the words "20 per cent."

3.** Page 29. Change 12th line under *2. Procedure* to read "Dissolve the combined sulfides in nitric acid and hydrochloric acid, evaporate to dryness, and redissolve the residue in hydrochloric acid and repeat. . . ."

4.** Page 37. In *section 1.4*, change final line to read: "1 ml. is equivalent to 0.05 mg. of fluorine."

5.*** Page 43. Change 2nd line under *2. Procedure* to read "dilute to the mark and mix thoroughly."

6.** Page 46. Change reference in the 2nd line to read "(Part I, Sec. XVIII, A, 3)."

7.** Page 53. Place parentheses about words "whole titer" in last two lines.

8.** Page 54. Change the figures "30.504" in the table following the word Bicarbonate to "61.008."

9.** Page 54. Place asterisk at the end of first sentence after table and insert footnote—"In routine practice a variation from 15 per cent in waters of less than 50 p.p.m. residue down to a 2 per cent variation in waters over 1,000 p.p.m. residue may be expected."

10.*** Page 63. Change opening part of first paragraph under *2. Procedure* to read "Measure 200 ml. of the sample into a 500 ml. Pyrex Erlenmeyer flask. Place 200 ml. of distilled water in another similar flask. Treat the contents of each flask in the following manner: Boil 15 minutes to expel free carbon dioxide. Add exactly 25 ml. of soda reagent and mix thoroughly. Boil 10 minutes, etc."

11.** Page 65. Add an asterisk after the word *made* in the sixth line and insert a footnote—"*A blank may be run using the indicator with freshly prepared ammonia-free water, titrating to a definite color, and deducting in all routine determinations the amount of acid used with the blank."

12.**** Page 70. Transpose the 7th and 8th lines from the bottom of the page.

13.*** Page 74. In *section 1.1* after the word "oxidized" add "as evidenced by the persistence of a faint pink color."

14.** Page 75. In last line of *section 2.2* change "6 *N* hydrochloric acid" to read "3 *N* hydrochloric acid."

15.** Page 86. In *section 1.3*, change the second sentence to read "To this add 140 g. of C.P. ammonium chloride and 350 ml. of C.P. ammonium hydroxide (sp.gr. 0.90)."

16.*** Page 87. In next to last line on page, change "0.9534" to read "0.8534."

17.*** Page 89. The drawing should show the water discharge lines leading to the catch basin cut off at least 1 in. *above* the top of the basin. This change is made so as to indicate equipment from which no back siphonage may occur.

18.*** Page 92. Change second line to read "Part II" instead of "Part I."

19.**** Page 110. In *section 1.4* the formula for sodium bisulfite should read "$NaHSO_3$."

20.**** Page 137. In the 7th line of *section 1.3*, change the words "oxidizing capacity" to read "oxygen consuming capacity."

21.** Page 140. In the drawing of the sampler, the edge of the lid should be shown flush with the outside of the sampler, i.e., $6\frac{1}{4}$ in. in diameter.

22.** Page 141. Change "mg." in the last line to "g."

23.**** Page 153. Subheadings in the table should read "Chloride in sea water (parts per million)" and "Dissolved oxygen in parts per million by weight."

24.** Page 177. Make figures in third line from bottom of page read "50 and 20 mm."

25.** Page 203. Delete "0.05" in line 7 and insert "0.5" to make read "0.5 per cent."

26.** Page 204. Make lines 7 and 8 of *section L* read "basic fuchsin (85 to 90 per cent dye content) in water."

27.*** Page 213. Change the figure "48" in *sections 2.2 and 2.3* to read "24."

28.** Page 221. Change "c.c." at top of table to "ml."

29.*** Page 225. Insert in line 2 of *section 1.2* after the word "bottle" the words "in presence of excess of crystals." It should also be noted that the original article on this method (ref. 1) refers to "ammonium carbonate-carbamate."

30.*** Page 227. Change "20" in line six to read "0.05."

31.**** Page 233. Substitute for *section 1.2* in its entirety the following: "1.2. Boric acid-borax buffer solution, pH 6.5. (a) Dissolve 12.4 g. boric acid (H_3BO_3) in distilled water and make up to 1 liter. This solution is approximately M/5 and has a pH of about 4.6. (b) Dissolve 3.8 g. borax ($Na_2B_4O_7$) in distilled water and make up to 1 liter. This solution is approximately M/100 and has a pH of about 9.2. (c) Add a sufficient volume, usually about 80 ml., of (b) to 1 liter of (a) to produce a buffer solution of pH 6.5."

32.*** Page 234. Change lines 1 and 2 of *section 1.1* to read "Dissolve 5.0 g. of potassium dichromate and 20 ml. of concentrated sulfuric acid. . . ."

33.** Page 261. Substitute in line two of *section F* the word "selective" for "specific."

34.** Page 264. In the list of specifications of ingredients, f4 should read "Certified by the Biological Stain Commission (Cert. No. CBg-3 (Coleman and Bell Co., Norwood, Ohio) or equivalent)."

35.** Page 269. Change the word "complete" in line 2 to "completed."

36.** Page 286. Change the first sentence under *Option 2* to read "Into clean wet bottles add approximately 0.02 to 0.05 g. of powdered sodium thiosulfate."

Users of the text should check their copy and enter all corrections. In the interest of all laboratory workers, it is important that any further errors be brought to the attention of the editorial committee at once so that corrections can be made in the fourth printing.

While all the corrections listed have appeared important to the editorial committee, particular attention is invited to items 27 and 31.

On page 213 of the text, a provision was inadvertently carried forward from earlier editions. Correction 27 indicates that a 24-, not 48-hour incubation period, should be used with secondary or confirmatory broth tubes. The correction brings this portion of the text in accord with the schematic outline on page 216.

Correction 31 relates to an amendment to R.D. Scott's chromate-dichromate standards for the estimation of residual chlorine. While this method is provisional (not standard), it has been widely used. The substitution of the boric acid-borax buffer eliminates the white precipitate which formerly developed in the standard tubes and is a definite improvement.

The current tendency toward examination of 100 ml. portions of water accentuates the need for precision in making up broth used for primary or secondary fermentation tests. It is feared that the failure of some workers to obtain concordant results in various 10, 50 and 100 ml. plantings may be due to failure to consider the need for reasonably exact concentrations of the food constituents. Concentrated lactose broth must be prepared so as to obtain a concentration of 0.3–0.5 per cent each of lactose and of peptone in the mixture after the sample has been added to the medium.

Definite plans have not been made for the preparation of a new edition (the ninth) but it will not be issued before 1942.

JOHN F. NORTON,
American Public Health Association
HARRY E. JORDAN,
American Water Works Association

Water and Sewage Works in Chile

By Leonardo Lira

THE last general census of the Republic of Chile, taken in the year 1930, gave a total population of 4,287,445, of which 2,119,621, i.e. 49 per cent, correspond to urban population, the balance of 2,167,824 being rural population. The total surface of Chile is 741,767 square kilometers, the density of population thus being 5.8 inhabitants per square kilometer.

The approximate urban population is distributed as follows:

In	4 cities with over 50,000 inhabitants	1,020,616
	10 cities between 20,001 and 50,000 inhabitants	350,793
	18 cities between 10,001 and 20,000 inhabitants	261,502
	21 cities between 5,001 and 10,000 inhabitants	152,046
In	162 villages between 1,001 and 5,000 inhabitants	334,664

Apart from these centers of population of greater importance, there are also 1,194 villages classified as rural, with a population between 201 and 1,000 inhabitants, their total being 464,704 inhabitants.

The Political Constitution of the State rules that the municipalities are obliged to attend to the sanitation of the populated centers within their territories. In the majority of cases, however, this constitutional mandate could only be carried out with State help, and finally the State, by law dated February 12, 1906, took charge of all these services. Since that date, the State has resolutely faced the problem of sanitation of populated centers and large sums of money have been appropriated yearly in the national budget for the construction of water and sewage works.

As a result of this policy at present there is *no town in Chile with*

A paper presented by Leonardo Lira, Director General, General Administration of the Utilities of Potable Water and Sewerage, Ministry of the Interior, Santiago, Chile. This paper is published to clarify impressions that may have been produced by parts of an abstract appearing on page 924 of the May, 1939, Journal of the A. W. W. A.

over 5,000 inhabitants which does not have its public water supply, and there is not a single city of over *10,000 inhabitants* which lacks a sewage system. Beside this, in the whole country, there are only 10 villages of over 2,000 inhabitants which still have no water supply system; but on the other hand there are 52 villages with less than 2,000 inhabitants who already have this service. As may be seen, not even the United States can show such a state of progress in respect to these utilities.

Expressing these figures as a function of the total population of the country, we get the following figures:

Percentage of population with water supply works	44.4
Percentage of urban population with water supply works	89.8
Percentage of population with sewage works	37.4
Percentage of urban population with sewage works	75.7

Form of Management

As a matter of fact, the great majority of public water and sewage systems have been constructed or improved by the State, which has reserved for itself their management until reimbursed of capital spent in their construction. For this reason, although the water works and sewage works are owned by the respective municipalities, it is the State which attends to their management through the "Dirección General de Agua Potable y Alcantarillado," a dependency of the Ministry of the Interior (Ministerio del Interior). This same office has the surveillance of the few works still managed by the municipalities, and the even fewer ones under private ownership.

The "Dirección General" mentioned has organized its task by grouping the different services in provincial nuclei, each nucleus being under the immediate management of an expert sanitary engineer. Six modern bacteriological and chemical laboratories have also been installed by this same office, strategically located throughout the country, for the permanent control of the quality of the water supplied by all the services.

It will surely be gratifying to the inhabitants of the United States to know that, in Chile, bacteriological examinations are made in accordance with the *Standard Methods* approved by the A. W. W. A. and A. P. H. A., and that the laboratories were installed by an American expert, Mr. Stalbird.

With such an administration, even the smallest villages have their water and sewage services under the control of an expert engineer,

and the waters are tested every week by a regional bacteriological laboratory, which attends to 20 or more services.

This system of organization has permitted the realization of a vast technical labor, highly specialized, in the management of water and sewage systems. Thus, apart from routine control of rapid and slow filtration plants, special studies have been undertaken, and are being continued, for the elimination of color and odor in the cities of Valparaiso and Quillota; the most successful processes have been studied for avoiding trouble from algae that grow in the settling tanks of the slow-filter plant of Riecillos; an investigation is being continued on the disturbance of coagulation in the rapid filter plant of Lebu, the water there having a high content of finely divided organic matter. Studies have been made with success for the correction of "red water troubles" in Valparaiso and Viña del Mar. Likewise, successful disinfection of the water supplied to Quillota has been made, based on chloramination, and the taste of the water supplied to Requinoa has been eliminated by the use of activated carbon.

Incidentally, it may be of interest to state that the apparatus for supplying chlorine to the water, used in Chile, were built by Wallace and Tiernan, and that the laboratories use culture media made by Difco Laboratories.

The results of this organization can best be followed from a perusal of the following data, taken from the Annual Report for 1938 issued by the head office:

Cities with water supply system in use	96
Percentage of metered services	87
Water supply systems with rapid filtration treatment and chlorine disinfection	8
Water supply systems with slow filtration treatment and chlorine disinfection	7
Water supply systems with sedimentation treatment and chlorine disinfection	2
Water supply systems using subterranean water and chlorine disinfection	34
Water supply systems using surface water and chlorine disinfection	22
Water supply systems using untreated subterranean water	10
Water supply systems using untreated surface water	13

Cities with sewage systems in use	36
Cities with sewage systems with sewage treatment (Imhoff Tanks, sludge dessication, percolating filters)	4
Bacteriological examinations undertaken during the year, as routine samples	11,178
Chemical examinations carried out	71

It is interesting to compare, as in table 1, these figures with the corresponding figures for the State of New York for public water supply, as given on pages 680 and 681 of the JOURNAL of the A. W. W. A. for April, 1939.

As regards laboratory control, there are carried out annually in Chilean services 11,178 bacteriological tests which are on semi-weekly samples for the greater number of services and on weekly

TABLE 1

Comparison of Water Supplies in New York State and Republic of Chile

	STATE OF NEW YORK		REP. OF CHILE	
	Number	Per cent	Number	Per cent
Number of towns or existing services	769		96	
Number of services with treatment (filter or disinfection)	379	49.5	73	76.0
Number of services with treatment by filtration	139	18	15	15.6

or fortnightly samples for the services at greater distance and lacking treatment plants. Figures for the State of New York show that the Health Department tested about 5,000 samples taken at the rate of from 4 to 24 samples per annum for each system. District laboratories tested monthly samples from 260 systems and only 95 systems have been submitted to the control of a local laboratory.

The Chilean systems using surface waters without treatment supply a total population of only 41,223 inhabitants. Of these systems, the chief one supplies the harbor of Tocopilla which has 18,000 inhabitants. Here the surface waters are taken directly in the heart of the Andes Range at an elevation of 3,800 meters above sea level, close to the international border line with Bolivia.

To the foregoing data, which are only a very brief résumé of the main results obtained in Chile with regard to the water supply and sanitation works for its populated centers, one must add the opinion

of foreign technical experts who have declared the services to be really efficient. Dr. Long, Delegate of the Panamerican Sanitary Office, has said: "The Chilean water supply systems are splendid, better than in any other Latin-American country. Fifty per cent of the Chilean population drink potable water, that is to say, pure and healthful. Chile has 57 cities (in 1937) where chlorination is used. Thus we cannot wonder that typhoid, dysentery and colic have disappeared."

On the other hand, already in 1931, the Second International Congress on Sanitary Technique and Urban Hygienics, held in Milan, had celebrated the solution adopted in Chile for getting the permanent attention of a sanitary engineer for the smallest villages.

Lastly, there should be noted a fact that demonstrates the degree of efficiency and training attained by the staff of engineers and experts of the Chilean services: The "Dirección General de Agua Potable y Alcantarillado de Chile" publishes an official Technical Review which contains only experimental work and original matter related to the management of its services. This Review, which is the first of its kind to be published in Central or South America, has been requested by, and eulogistically received by, the various engineering societies of the different American countries.

Discussion by *C. R. Cox.** The brief review of the control of public water supplies in Chile prepared by Mr. Lira is worthy of careful consideration. It indicates what can be accomplished in a relatively short period of time when concerted effort is made to provide organized control of public water supplies by a central agency having adequate authority and public support. It will be apparent that the control procedures discussed by Mr. Lira are in advance of many countries throughout the world. This accomplishment is of special interest to the New York State Department of Health because the work was organized by J. A. Stalbird during the period 1925–1931 before he joined the staff of this Department as District Engineer.

The discussion is too brief to indicate the interrelations between local and central government agencies. It is evident, however, that the strong central government in Chile has facilitated the organization of a nation-wide program which would be difficult to achieve were it necessary to convince local officials of the necessity of ade-

* Chief, Bureau of Water Supply, State Dept. Health, Albany, N. Y.

quate treatment and the control of various water supplies and to train locally the necessary technical operators capable of providing laboratory control of individual supplies. The achievements in Chile, therefore, reflect a proper realization of this situation through the utilization of a central and six branch laboratories to which samples are shipped for examination at frequent and regular intervals. In this way specific supplies of an unsatisfactory nature are brought to the attention of properly trained sanitary engineers to guide them in their inspection and control program.

The references in Mr. Lira's discussion to the control of public water supplies in New York State must be viewed in the light of the limited published data which were available to Mr. Lira. Table 1 of Mr. Lira's discussion, for instance, indicates that about one-half of the public supplies in New York State are not subject to treatment, which, statistically, is a correct statement of the situation. The conclusion should not be reached, however, that the remaining supplies need to be subject to treatment by either chlorination or by filtration and chlorination because the great majority of the untreated supplies are secured from wells or springs which yield water of safe, sanitary quality not needing treatment. It is impossible, therefore, by the comparison of the few data shown in table 1 to formulate ideas concerning the relative situations in New York State and in Chile.

In order, therefore, that New York State data may be used as a background in reviewing the praiseworthy accomplishments in Chile, it seems pertinent to record certain information.

Based upon the 1930 population figures and information as of May, 1939, there are in New York State a total of 772 supplies serving 1,149 communities with a total population of 11,599,000. This represents 93 per cent of the total population in the State as of 1930. Of the 772 separate supplies, 402 were secured from wells, springs or infiltration galleries, 323 from streams, lakes or reservoirs, and 47 from combinations of both ground water and surface water. This clearly indicates the great importance of spring and well supplies serving the smaller communities in New York State where, fortunately, treatment is not needed in the majority of instances.

Obviously springs and wells having limited yields are not the usual sources of water supply for larger communities. It is not surprising, therefore, that the majority of the larger communities are served with treated surface supplies. Thus approximately 9,135,000 out of the total of 11,599,000 served by public supplies receive treated

water. Because of the existence of numerous mountain lakes and relatively clear mountain streams draining forest covered watersheds in New York State, the number of filtered surface supplies is lower than might be expected. This is indicated by the fact that only 29 supplies are treated by slow sand filters, 112 supplies by rapid sand filters, and 380 supplies are treated by chlorination alone.

Laboratory control over public water supplies in New York State is predicated upon the basic policy that the problem is primarily local in nature and represents a duty of the local officials insofar as their resources permit. Every effort has been made, therefore, to encourage municipalities and water companies to employ expert chemists to supervise the operation of large plants or to employ consulting chemists for less frequent advice. Consistent with this policy, about 95 filtration plants are subject to detailed laboratory control involving the examination of many samples each day, and consulting chemists supervise the operation of 37 other plants.

Thirty counties in the State are served by county laboratories, the services of which are available to municipalities and water companies. These laboratories examine approximately 14,000 samples of water each year, about 6,000 of which are collected from municipal water supplies in the respective counties. In addition the laboratory of the State Department of Health examines about 7,500 samples of water each year, about 5,000 of which are collected from public water supplies, the remaining samples being from private supplies and miscellaneous sources.

Too much emphasis, however, cannot be placed on control programs of this nature unless the work is supplemented by and coordinated with frequent inspections by sanitary engineers who are in position to confer with local water officials and offer the facilities of the State Department of Health in solving pressing problems. In this connection approximately 1,400 inspections and conferences were held by the engineers of the 20 district offices of the Department during 1938.

Accomplishments to date indicate the need for more frequent examination of samples of water from the respective supplies and especially for the organization of local laboratory control. For this reason special stress is being placed upon the training of operators in charge of the supplies subject to treatment in order that they may be able to provide adequate supervision over the treatment processes. Schools for such operators have greatly facilitated the extension of

local laboratory control to communities unable to employ graduate chemists or engineers.

In the last analysis, of course, the centralized control of public water supplies only serves to coordinate, standardize and stimulate adequate local control of the respective supplies. Water purification equipment is no more reliable than the manner in which it is operated, as shown by the fact that conscientious and effective control of inferior equipment may produce results superior to those secured from less effectively operated equipment of modern design. For instance the availability of chlorination equipment does not represent necessarily a definite accomplishment until such equipment is continuously and effectively operated under suitable local supervision including frequent ortho-tolidine tests, whereby adjustments are made for frequent fluctuations in quality of the water, which fluctuations will not be disclosed by the periodic examination of samples in a central laboratory. Control programs, therefore, cannot be altogether effective unless local officials are fully conscious of their responsibilities.

In conclusion it seems appropriate to emphasize the point that more discussions of the character prepared by Mr. Lira are needed in order that everyone may benefit by the progress being made in various countries. Mr. Lira and his associates in Chile are to be commended upon the accomplishments reviewed in his article.

Joint Administration and Collection of Water and Sewer Accounts

Committee Report

AT THE 1938 convention of the A. W. W. A. there was held a preliminary discussion of sewer rentals or service charge method of financing sewerage works. This subject is becoming of increasing interest to water works men because of the prevailing practice of using water rates, water consumption, or water demand as a basis for determining the sewer service charge. Furthermore, the billing and collection facilities of the water works departments are being used for sewerage accounts.

For these reasons it seemed highly desirable for this association to undertake a special study of problems involved in the joint administration and collection of water and sewer service charges. Early this year a temporary committee was formed to assist in scheduling further discussion of this important topic. This report of the committee is presented, not as a completed study, but more as an introduction to a full study to be subsequently completed so that our members may be kept informed of developments and experiences on special problems of sewerage administration that affect and are of interest to water works men.

Much valuable information has been received from a number of cities and it is felt that further data should be collected and included in the official records of this association, where it will be available for the use of members. Methods of billing, sample forms of bills,

A committee report presented on June 14, 1939 at the Atlantic City Convention by: Frank O. Wallene, Chairman of the Committee and Director of Public Utilities, Cleveland, Ohio; William M. Piatt, Consulting Engineer, Durham, N. C.; Laurence G. Lenhardt, Commissioner of Public Works, Detroit, Mich.; Leonard N. Thompson, Gen. Supt. & Engineer, Water Department, St. Paul, Minn.; Robert A. Allton, Consulting Engineer, Div. of Sewage Treatment, Columbus, Ohio; and W. F. Tempest, Repr., Highways and Municipal Bureau, Portland Cement Assn., Chicago, Ill.

number and size of meters, detailed rate schedules and many other important items are covered.

In some cases the responsibility for administration of the sewerage utility was handed to the water works department with little or no warning. The water works executive was then confronted with problems for the solution of which there was but meagre, scattered information. In the interest of advancing municipal improvement and economy close cooperation between the two essential complementary services, water works and sewerage, is highly desirable. Neither of these services is fully appreciated by the average citizen, who is inclined to think of them as general functions of government rather than of individual utility service.

It is the opinion of many in charge of water works that the combined billing of water and sewer service, when the charge is a function of water consumption, will result in reduced use of water and consequently lower income if rates remain the same. To some extent this is to be expected in the larger cities, but in smaller communities the improved sanitary facilities may actually result in increased water consumption.

The sewer service charge is being considered more and more as a means of financing wholly or partially the cost of improvements and extensions. The accounting and collection procedure is a responsibility that water works men will be called upon to carry. Free exchange of information and ideas on this subject will be a great help in smoothing the path of the water works executive faced with these new problems.

To facilitate the collection of both water and sewer charges the committee finds a definite need for more education of the general public. All cities are not in agreement on the need for an educational campaign other than the usual notice of change and publication of the ordinance in the local newspaper.

When, however, an additional item is added unexpectedly to the water bill, there are naturally a number of complaints and for some time at least collections will probably be difficult. Fortunately, the sewer charge is relatively small for individual accounts and works no hardship on even the poorer families.

One of the most difficult problems and fundamental to successful operation is the establishment of an equitable rate schedule. While the burden of responsibility for the rates should remain with the sewer division, it is highly desirable that they be made with the advice and consent of the water works executive.

In the case of Minneapolis and St. Paul, a long and careful study was made of the problem. As a result of this exhaustive study the general requirements and collection procedure were embodied in a resolution adopted by the Metropolitan Sanitary District, September 26, 1938.

In Minneapolis the charge is based on the combined consumption of city and well water and the monthly rates per 100 cu. ft. are as follows:

For First 100,000 cu.ft.	$.028	per 100
For Next 200,000 cu.ft.	.015	per 100
For Over 300,000 cu.ft.	.0075	per 100

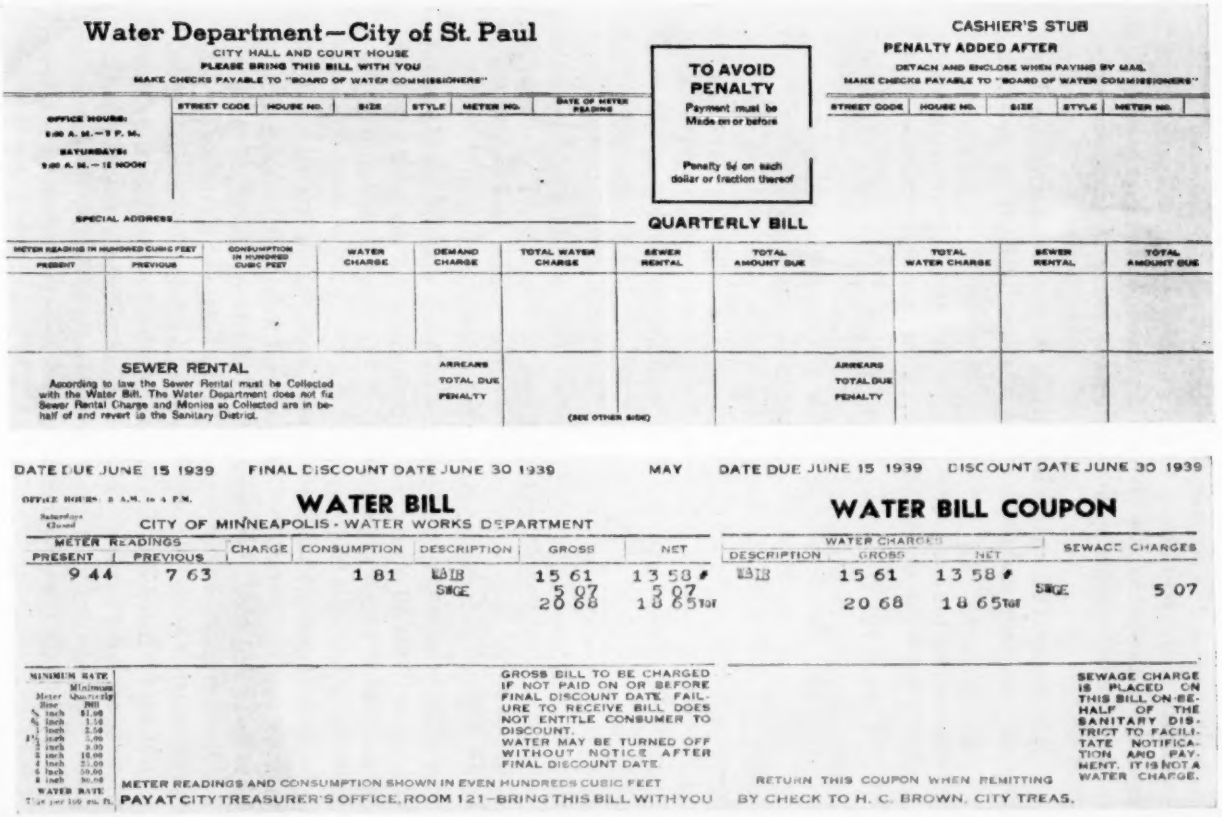
In St. Paul the sewer service charge is based on the size of the meter according to the following schedule:

Meter Sizes	Annual Charge
$\frac{5}{8}$-inch	$2.40
$\frac{3}{4}$-inch	4.00
1 -inch	8.00
$1\frac{1}{4}$-inch	12.00
$1\frac{1}{2}$-inch	18.00
2 -inch	36.00
3 -inch	75.00
4 -inch	180.00
6 -inch	360.00
8 -inch	630.00
10 -inch	900.00
12 -inch	1,350.00

In both cities commercial bills are sent out monthly and domestic bills are sent out quarterly, and in both cities the charges are handled by the water works departments. (The form of commercial and domestic bills is shown in figure 1.)

In 1935 a comprehensive ordinance was approved for sewerage improvements in Detroit. This provided for construction, financing, bond issue and form, rate schedule, and collection. This ordinance sets the sewerage rate at $0.195 per thousand cu.ft. of water used, but in another section under "Bond and interest redemption fund" it provides for revision of the rates if necessary to produce the amount required for bond service. The ordinance places the sewerage system under the jurisdiction of the Commissioner of Public Works of the City of Detroit. No experience can yet be reported on the operation of the ordinance.

Los Angeles, California, has urgent need of sanitary improvements and the city engineer has worked tirelessly in a sincere effort to find the most practical means of financing. He has recommended a



Water Department—City of St. Paul

CITY HALL AND COURT HOUSE
PLEASE BRING THIS BILL WITH YOU
MAKE CHECKS PAYABLE TO "BOARD OF WATER COMMISSIONERS"

OFFICE HOURS:
8:00 A. M.—3 P. M.
SATURDAYS:
8:00 A. M.—12 NOON

STREET CODE	HOUSE NO.	SIZE	STYLE	METER NO.	DATE OF METER READING

SPECIAL ADDRESS

TO AVOID PENALTY
Payment must be Made on or before

Penalty 5¢ on each dollar or fraction thereof

QUARTERLY BILL

METER READING IN HUNDRED CUBIC FEET PRESENT	METER READING IN HUNDRED CUBIC FEET PREVIOUS	CONSUMPTION IN HUNDRED CUBIC FEET	WATER CHARGE	DEMAND CHARGE	TOTAL WATER CHARGE	SEWER RENTAL	TOTAL AMOUNT DUE

SEWER RENTAL

According to law the Sewer Rental must be Collected with the Water Bill. The Water Department does not fix Sewer Rental Charge and Monies so Collected are in behalf of and revert to the Sanitary District.

ARREARS
TOTAL DUE
PENALTY

(SEE OTHER SIDE)

ARREARS
TOTAL DUE
PENALTY

TOTAL WATER CHARGE	SEWER RENTAL	TOTAL AMOUNT DUE

CASHIER'S STUB

PENALTY ADDED AFTER

DETACH AND ENCLOSE WHEN PAYING BY MAIL
MAKE CHECKS PAYABLE TO "BOARD OF WATER COMMISSIONERS"

STREET CODE	HOUSE NO.	SIZE	STYLE	METER NO.

DATE DUE JUNE 15 1939 FINAL DISCOUNT DATE JUNE 30 1939

MAY

DATE DUE JUNE 15 1939 DISCOUNT DATE JUNE 30 1939

OFFICE HOURS 8 A.M. to 4 P.M.
Saturdays Closed

WATER BILL

CITY OF MINNEAPOLIS - WATER WORKS DEPARTMENT

METER READINGS PRESENT	METER READINGS PREVIOUS	CHARGE	CONSUMPTION	DESCRIPTION	GROSS	NET
9 44	7 63		1 81	WTR	15 61	13 58 #
				SWGE	5 07	5 07
					20 68	18 65 Net

WATER BILL COUPON

WATER CHARGES DESCRIPTION	GROSS	NET	SEWAGE CHARGES
WTR	15 61	13 58 #	
			SWGE 5 07
	20 68	18 65 Net	

MINIMUM RATE

Meter Size	Minimum Quarterly Bill
¾ inch	$1.00
1 inch	1.50
1½ inch	2.50
2 inch	3.00
3 inch	10.00
4 inch	21.00
6 inch	50.00
8 inch	80.00

WATER RATE
Ten per 100 cu. ft.

GROSS BILL TO BE CHARGED IF NOT PAID ON OR BEFORE FINAL DISCOUNT DATE. FAILURE TO RECEIVE BILL DOES NOT ENTITLE CONSUMER TO DISCOUNT. WATER MAY BE TURNED OFF WITHOUT NOTICE AFTER FINAL DISCOUNT DATE.

METER READINGS AND CONSUMPTION SHOWN IN EVEN HUNDREDS CUBIC FEET

PAY AT CITY TREASURER'S OFFICE, ROOM 121—BRING THIS BILL WITH YOU

RETURN THIS COUPON WHEN REMITTING BY CHECK TO H. C. BROWN, CITY TREAS.

SEWAGE CHARGE IS PLACED ON THIS BILL ON BEHALF OF THE SANITARY DISTRICT TO FACILITATE NOTIFICATION AND PAYMENT. IT IS NOT A WATER CHARGE.

FIGURE 1

"pay-as-you-go" basis with sewer service charges placed on the water bills. It is anticipated that there will be some educational work needed to assure public support, but the need is imperative and the means of financing practicable and fair to all, so apparent opposition should be overcome.

Taxpayers are becoming increasingly conscious of the cost of government. It is important that in striving for economy they be made to realize the necessity for the essential domestic services so that ill-considered economy cuts will not impair the efficient functioning of the public utilities departments. The income of such departments is best assured by a service charge, which to a large extent removes the precarious conditions resulting from complete dependence upon budgetary allotments. In the case of water works and sewerage such charges are conveniently handled under one system of billing and collection. When this is done, one of the following methods is commonly employed:

1. One bill and one amount (water charge includes sewerage, but not divided on bill).
2. One bill and one amount, but bill states charges for water and sewer combined.
3. One bill, but two separate amounts (one for water and one for sewer service).
4. Bills sent out by water department, but separate bills for water and sewer service.

Information was obtained from 26 cities with populations ranging from a few thousand to the larger cities including Detroit with population of 1,641,000, Minneapolis with 500,000, and St. Paul with 270,000. Thirteen of the 26 cities replying have a Public Works Department. Six of these reported this department responsible for water works and 10 reported Public Works Department responsible for sewerage. Eleven cities reported that the water works department makes sewer service charge. In a few cases another department of the government makes the actual charge and designates the water department as the collection agency. Twelve of the cities reporting indicate combined administration of water works and sewerage departments. In reporting on whether the owner or the user paid for water and sewerage service, 17 reported the user paying for water and 15 reported the user paying for sewer service.

Replies to the question of percentage of population paying minimum and maximum amounts were generally incomplete, but the

FIGURE 1

information that was given indicates there is relatively a small percentage of the total number of customers paying the maximum rate.

Information on the average water bill for a $\frac{5}{8}$-inch connection indicates that this utility service is being provided at very low cost.

In changing to a form of billing for which a specific charge was made for sewer service based upon the water consumption, the cities generally informed the water users of this change in policy either by advance notice with previous water bills or by newspaper publicity in connection with publication of the revised ordinance. Very few conducted any special educational campaign previous to the adoption of new methods. Some believe this is not necessary, but the majority seem to favor some form of campaign and believe that it would materially reduce subsequent objections when the bills are received for sewer service as well as for the water consumed.

The general reaction to the new form of billing appears to have been favorable. In one or two cases the reaction was unfavorable and objections came from certain organized minorities and others not familiar with the rate basis and especially renters who had previously been accustomed to the owner paying the sewer charge.

In reporting on the changes involved in the water department adopting the collection of sewerage accounts the consensus of opinion was that the costs would be increased for billing and in some cases additional clerks would have to be added. It is believed that the reports, however, did not consider the cost of operating two separate billing and collection agencies, but rather interpreted the question to refer particularly to their own department. One reply stated definitely that the final cost would be less, estimating approximately 30 per cent reduction. Estimates of those reporting increased costs indicated that it would not be a serious burden and could very well be taken care of in the regular budget.

The effect of mechanical billing as an aid in the reduction of the cost could not definitely be determined as many were already using such equipment and it would mean merely greater use of the machines. The use of addressing machines and accounting machines appears to be quite general for the smaller communities as well as the large cities.

To result in a uniform flow of work and to facilitate collection, the billing periods are usually staggered or the city divided into sections and the bills sent out at different times during the quarterly billing period. The tendency seems to be to use one form of bill with the

two items for water and sewer service separated so that the customer knows precisely what he is paying for each.

It is the general practice not to permit payment of one item alone, although in some extreme cases partial payments of the total bill will be permitted where the customer is in distress.

As indicated by previous surveys, the percentage of delinquent accounts is comparatively small for the water and sewer utilities.

Fifteen cities reported that the sewer service charge covered all items included in capital costs, maintenance and operation. Four reported that the service charge was for operation and maintenance only, and in one case was reported to be for capital costs alone.

The general understanding of the term "sewerage system" appears to be that it refers to all collection lines, interceptors and appurtenances in the system proper, while "sewerage works" covers the entire plant from the collection system to final clarification. "Sewage disposal" is understood generally to mean exclusively the treatment or disposal plant and some include in this the interceptors.

There appears to be practically no exemption of the sewerage charge with the exception of certain communities where county hospitals and in one case churches were exempt. In another, clear water diverted to storm sewers or water used for irrigation purposes was exempt from sewer service charge.

Quite generally, when adjacent communities are served, the rate is considerably greater than for those receiving the same service within the city limits. Many charge double the city rates.

A comprehensive chart of replies to questionnaires will be published at a later date. This will be divided into sections and will present in detail the information outlined in this report and subsequent material received in response to further inquiry.

F. O. WALLENE, *Chairman*
WM. M. PIATT
L. G. LENHARDT
L. N. THOMPSON
R. A. ALLTON
W. F. TEMPEST

Experiences in Establishing Sewer Rental Charges

By F. O. Wallene

THE purpose of this paper is to set forth the experiences in the city of Cleveland in setting a rate for sewage service charges, so that the record may be helpful to others who have not passed through this experience.

The administrative problems of the Director of Public Utilities of the City of Cleveland are manifold and complex. They involve the administration of three divisions, namely: Water and Heat, Light and Power, and Sewage Treatment and Disposal. They represent in all a total investment of approximately 110 million dollars. Practically all branches of engineering, as well as other professions are involved. There is a body of 2,500 employees in the department, who in themselves represent a problem in "human" engineering. In addition to the usual construction work, and at the moment the building of a new five and one-half million dollar light plant, as well as tuning up and placing in operation the new twenty-million-dollar sewage plants, there is a question of rate making for the three divisions, and the necessary public relationship problem therewith. No doubt you will agree there is plenty to keep one busy. But all of these problems are just so much "mouse meat" when compared to the problem of joint collection of water and sewerage charges.

To Mr. Average Citizen there is only one subject more disagreeable to think or talk about than that of sewage, and that is—payment for sewage disposal. Like the housekeeper who sweeps the dirt under the rug, he is not concerned with it as long as it is out of his sight, and to be constantly reminded of it by being billed for its disposal; well, that is what some people refer to as "political dynamite." No amount of publicity, educational or otherwise, will completely recon-

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A paper presented on June 14, 1939 at the Atlantic City Convention by F. O. Wallene, Director of Public Utilities, Cleveland, O.

cile him to paying the cost for this new service. In Cleveland, practically every known means of educational publicity has been resorted to. The newspapers have aided in a substantial way. The radio, as well as group meetings, have been utilized in an effort to bring the people to a realization that this new service is worth the price they must pay. The most effective argument probably is the statement that the sewage disposal plants are fundamentally the same as the water purification plants, differing primarily in the quantity of heavy pollution substances they are capable of removing. Further, it has been shown that the operators of these plants are guardians of public health in a way similar to the medical profession, the main difference being that the department functions as a "preventive device" whereas the medical profession, in the main, uses "curative means." The public must be brought to realize that large concentrations of pollution in this modern world of ours have overwhelmed nature's methods of purification by dilution, aeration and sun's rays, and that scientific means must be used. This obviously entails increased costs that must be met.

Some refer to sewerage service as a utility as the name applies to electric light and power, telephone and gas, although some of the court decisions have not concurred in this definition. No doubt as time goes on the opposition will diminish, and particularly so if the citizens can be induced to visit the plants so as to obtain a first-hand picture of the service that is being rendered. It is practically impossible for any administrator to bring people to realize the benefit derived from this service without the full cooperation of the legislative body of the city, and this in itself presents an extremely difficult problem. Cleveland's legislative body consists of 33 members who come up for re-election every two years. They are, of course, continually harassed by their constituents in their various wards and it must be admitted that their problem is not at all simple when it comes to convincing people that even a sum of $1.00 per quarter added to their water bill is a just charge and should not be complained of. This happens to be the average charge of 75 per cent of our consumers, and includes the average size as well as the small home.

Cleveland's plants on the new and mechanized process basis of operation were started in July of 1938, at which time the Council adopted water consumption as the basis of charge and means of producing revenue for the operation of the plants. This was done under authority of a state statute. After careful study by the Ad-

ministration, and concurred in by the Council, an ordinance was passed setting up a rate of 40 per cent of the water bill as a sewer service charge, applying to the city of Cleveland proper. At the same time an ordinance was passed for the suburbs fixing a charge of 75¢ per 1,000 cu.ft. The city's rate is approximately 32¢ per 1,000 cu.ft.

Many public hearings were held before final passage. Generally speaking, the opposition came from a very small group of people, many of whom freely admitted that they would prefer a continuance of the pollution of Lake Erie (which is the water supply of the city) in preference to paying anything for sewage disposal. As this attitude became known, council members were less impressed with the opposition as they well realized the need for the service and therefore acted accordingly in spite of the continued threats of political extermination at the next election. Industry as a whole had practically no objection, probably for the reason that men in industry had made a study of the situation and concluded that the proposed system of charge was fair and equitable. Some of the council felt that the operating money should be obtained from real estate tax, although they agreed that fundamentally, charges should be proportioned to the service rendered, and this could only be accomplished on the basis of volume of water used and converted sewage.

Primary Treatment Paid by Real Estate Taxes

Up to July of 1938 the plants were operated as "primary" treatment plants only and the money was derived from real estate taxes. The betterments that were installed called for an increase in personnel and power requirements, as well as supplies including chemicals, etc. Obviously some difficulty was experienced in our efforts to explain even to the most enlightened citizen that the cost of operation of the new plants would be much greater, but that the improved efficiency was worth the added expense. To complicate matters further, the yearly budget provided for funds from real estate taxes sufficient only to operate the plants up to July 1. As a result we found ourselves with three large plants ready for full operation, and no money to proceed with.

Water billing is on a quarterly basis, and no charge could be made prior to the effective date of the ordinance, namely, July 17, 1938. Furthermore, only water converted to sewage that actually reached the disposal plants could be charged for. Some areas in the low-level

district discharge their sewage directly into the river. Ultimately this will be picked up and pumped into the regular sewage system. Thus they are what might be termed "exemptions," some of which were known and others unknown, due to the lack of records in this very old part of the city. This also brought up the argument as to why some people should escape the charges even though the city had not made provision for the treatment of their sewage.

The Department found it necessary to direct the sewage gradually into the new plants in step with the revenue taken in (at the same time, gradually increasing personnel, power use and purchase of supplies), with the net result that the plants were not placed in full operation until April 1, 1939. In the meantime, many of the council members kept asking for information as to cost of operation, revenue obtained, expressing the belief that the rates could be reduced, and thus reduce the pressure of their constituents in the approaching election.

Rates for Suburbs Set Up

Accounting figures were obviously incomplete, and this produced the further hazard of having a new rate set, based on misleading data that would make it impossible to operate the plants in the manner for which they were designed. Further aggravation in the situation exists in Metropolitan Cleveland where there are a larger number of suburbs using Cleveland's sewerage system, with which there is difficulty setting up contractual relationship considered fair to both sides. The Water Department serves the whole Metropolitan Area. Some of these suburbs are served wholesale through master meters. A rate ordinance has just been drawn up which is believed now acceptable to both sides. In the case of the direct metered suburbs, further complications arise in connection with the collection of delinquent sewerage service charges. In the city proper, sewerage charges can be made a lien on the property, although this is not true of water. In the suburbs there is some question as to whether the lien law can be applied, except perhaps by contractual relationship with the respective political sub-division. The Department is also confronted with the claim by one of the suburbs that it has practically no infiltration, and also that it has a separate storm sewer system as distinguished from the balance of the territory. The suburb claims it is entitled to a lower rate, but the further disturbing fact is that this suburb which claims a lower rate is the "silk stocking" area,

which can better afford to pay than others populated by the poorer people.

The question of water used for lawn sprinkling has also been considered to a great extent. The theory has been followed that payments in the wintertime will be less than they should be, while in the summer months of sprinkling the cost would be somewhat greater, but that they would average out at the end of the year. Where the item is of considerable proportion, the Water Department installs a second meter for re-registering water used for this purpose, deducting this reading from the main meter. The same thing is done with industry where exemptions can be justly claimed, such as railroads, breweries, ice plants, etc.

Consumers to Pay Only Operation Charges

The city of Cleveland's charge to its own consumers contemplates operation charges only, interest and debt retirement charges being obtained from real estate tax. Thus the vacant lot pays fixed charges, and only when the building is erected and water is used on the premises, does it pay its proportion of operation costs. In the event of a vacancy of a building and discontinuance of use of water, the operation charge ceases, but the fixed charges remain. The rate to the suburbs, however, includes a proportional share of interest, depreciation, plus operation charges on the basis of volume of water used. This theory of charge has been generally accepted as fair and proper. Fortunately all water connections are metered, making this basis of charge practicable. There has been no thought, therefore, of having charges to be based on fixtures, population, etc., as has been the case in other localities. Using the basis of water consumed as a charge has also made it possible to include it on the water bill as a separate item, thus effecting considerable saving in accounting costs. The only disadvantage has been the criticism and failure on the part of Mr. Average Citizen to realize that his water bill has not been increased 40 per cent, but that it is a charge for an entirely new, separate and distinct service.

To avoid a continuance of debate and pressure from the opponents of the system set up, the Department is now attempting to set up a rate ordinance that provides a formula for an annual adjustment of the rate, based on the previous year's operation. For the city of Cleveland proper, this would include operation costs only, leaving interest and debt retirement on the general tax, but providing: first,

sufficient money for proper operation; second, an excess amount to meet unforeseen conditions or plant improvements; and third, that charges should be proportionate to the service rendered throughout the metropolitan area. The same plan would apply to the suburbs. We have made sufficient progress to believe that this can be worked out in a practical way, meeting all conditions. The difficulties encountered in Cleveland are not necessarily peculiar to this particular city, but probably appear in varying forms in all of the larger cities.

To the Administrators, let me say, that educational publicity is vitally important, and particularly so with the members of their legislative body. They have the very difficult problem of educating their constituents probably at the moment when they are asking for their political support. At the same time these same legislators should recognize that the administrator is not a magician and therefore unable to operate the plants properly with deficient income. It calls for cooperation to the fullest extent. Both sides need to realize that financially starved plants, of which there are many, cannot possibly be operated efficiently.

Licensing Water Works Employees

Status at end of 1939

Staff Report

THE American Water Works Association has, by acts of its various Sections and by specific resolution adopted by its board of directors, evidenced its support of licensing water works operators. The resolution of the board, adopted at its annual meeting in January, 1937, is as follows:

"The board of directors of the American Water Works Association realizing the importance of efficient operation of water purification plants as a protection to public health, endorses licensing, certification, or some form of regulation of water purification plant operators and supervisors as a means of preventing the employment of incompetent personnel and assuring some continuity of employment of efficient personnel. Sections of the A. W. W. A. are hereby encouraged to engage in voluntary licensing."

Committee activity of the various Sections has been carried on with continued vigor. The committee of the Association, under the leadership of Prof. L. V. Carpenter, has terminated its activities, but it is the current practice of the headquarters staff to maintain full information concerning the subject. Copies of various laws, regulations and plans are on file and reference may be made to this material by any member who has an interest in the program.

In the early part of 1939, while the various state legislatures were in session, an inquiry was directed to the various State Sanitary Engineers which sought to ascertain what developments were in progress. In due time, after all regular legislative sessions had closed, the replies were consolidated into tabulated form. This material has been reviewed by each State Sanitary Engineer and represents, in condensed form, the situation as it exists as of November 1, 1939. Fourteen states now either license or "certify."

It is to be hoped that all persons concerned will continue their

active interest in the licensing or certification of responsible employees in water works plants. There should be no debate over the value of trained, competent personnel in water supply operations. No reasonable effort should be spared which tends to require that the operation of water supply works be in the hands only of such persons as have demonstrated their fitness for the work and are protected in their tenure of employment by regulation or by law.

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See tabulation on next page.

WATER PURIFICATION PLANT OPERATOR LICENSING—1939

STATE	LIC. OR CERT. OF OPERATORS AS OF 1939	PLAN	CLASSIFICATIONS AND REMARKS
Alabama	No		Not contemplated
Arizona	No		Operators interested. Ariz. Sew. & Water Works Assn. has committee
Arkansas	No		Plans for voluntary licensing under way
California	Certify	Voluntary	Certification under the Calif. Section, A. W. W. A.
Colorado	No		Bill providing for licensing before 1939 Legislature did not pass
Connecticut	Certify		Qualifications of operators subject to approval of State Health Dept., by law. Rules for qualification are set up. System is satisfactory
Delaware	No		Not contemplated
Florida	No		Fla. Section, A. W. W. A. has committee investigating
Georgia	Certify		Certification plan satisfactory. May lead to licensing later
Idaho	No		Being considered
Illinois	Certify		"Certificate of competency" issued by state. 3 grades: (A) 5 yrs. exp. or educational qualification; (B) 3 yrs. exp. or educational qualification; (C) 1 yr. exp., chlorination treatment only
Indiana	No		Bill before Legislature failed of passage
Iowa	No		Some discussion, but no definite plan
Kansas	No		Some discussion, but no definite plan
Kentucky	No		Regulation passed, but State Health Dept. personnel not available
Louisiana	No		Being considered
Maine	No		Not contemplated
Maryland	No		Some discussions, but no definite steps
Massachusetts	No		Not contemplated
Michigan	License	Regulation of Dept.	Confirming legislation before the '39 Legislature failed of passage

Minnesota	No		Not contemplated
Mississippi	No		Not contemplated
Missouri	No		Considerable interest in it
Montana	No		Montana Section, A. W. W. A. interested. Legislation may be introduced in '41
Nebraska	No		Not contemplated
Nevada	No		Not contemplated. Legislation is available
New Hampshire	No		Not contemplated. Legislation is available
New Jersey	License	Statutory	Includes all persons in direct general charge of water supply systems, as well as persons in charge of water purification plants. Several classifications based on type of treatment, source of supply and capacity of plant
New Mexico	No		Not contemplated
New York	Certify	Statutory in effect	Certificate of qualification issued by State Health Dept. To operators of publicly owned plants only. Three grades of operators: (I) Serving more than 20,000 pop., or filter plant serving more than 10,000; (II) Serving 5-20,000 pop., or filter plant serving less than 10,000; (III) Serving less than 5,000 pop.
North Carolina	No		Passage of necessary legislation has been attempted unsuccessfully. Voluntary certification being seriously considered
North Dakota	No		N. D. Water & Sew. Works Conference has committee working
Ohio	Certify	Regulation of Dept.	Certificates issued in 4 grades: (A) Shall have an Engineer's license and six years experience; (B) Three to six years exp. depending on educational qualifications; (C) Three to six years exp. with lower educ. qualif.; (D) One year experience
Oklahoma	License	Voluntary	Just started. Handled by State Health Dept.
Oregon	No		Pac. N. W. Section, A. W. W. A., has committee studying the subject
Pennsylvania	No		Has been actively discussed for several years
Rhode Island	No		Not contemplated

WATER PURIFICATION PLANT OPERATOR LICENSING—1939—*Concluded*

STATE	LIC. OR CERT. OF OPERATORS AS OF 1939	PLAN	CLASSIFICATIONS AND REMARKS
South Carolina	Certify	Voluntary	Certification by S. C. W. W. Assoc., approved by State Health Dept.; 4 grades: (A) 8 yrs. exp. or certain equivalents; (B) 5 yrs. exp. or certain equivalents; (C) 3 yrs. exp. or certain equivalents; (D) Must be employed by a water plant, and capable
South Dakota	Certify	Voluntary	Certificates issued by S. D. Water & Sew. Works Conf. 3 grades
Tennessee	Certify	Voluntary	Certification under the State Health Dept.: (A) Filtration, more than 15,000 pop.; (B) Filtration, pop. 2-15,000; or chlorination, more than 15,000; (C) Filtration, less than 2,000; or chlorination, less than 15,000; (D) No treatment
Texas	License	Voluntary	License issued by State Health Dept. in 3 grades, depending on experience and examination passed
Utah	No		Not contemplated
Virginia	No		Not contemplated
Vermont	No		Not contemplated
Washington	No		Pac. N. W. Section, A. W. W. A., and others, are working on the matter
West Virginia	License	Statutory	Previous regulation of State Health Dept. strengthened by new law ('39) which specifically authorizes it. Certification in 3 grades depending on type of treatment and population served
Wisconsin	No		Proposed legislation failed to pass
Wyoming	No		Not contemplated

ABSTRACTS OF WATER WORKS LITERATURE

Key. 31: 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31:** 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, ★[ ]★, is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows. *B. H.—Bulletin of Hygiene (British); C. A.—Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R.—Water Pollution Research (British).*

WATER QUALITY STANDARDS

Composition of Water and Interpretation of Analytical Results. ATILIO A. BADO. Inspector-General of Public Health Laboratories. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 13 (Jul. '39). Reference is mainly to conditions in Argentina. Application of uniform hard and fast limits for permissible concentrations of all different ions over the whole of so vast and diversified an area, is not considered practical. Reviewing in turn the analytical findings deserving of comment, *total mineral content* is found to vary from less than 100 to more than 2000 p.p.m.: 1500 p.p.m. is perhaps the upper limit of what is desirable. *Hardness* may conveniently be classified as (a) not exceeding 100 p.p.m., (b) from 100 to 200 p.p.m., and (c) over 200 p.p.m. The corresponding percentages of surface supplies are 66, 17, and 17; and of ground water supplies, 44.8, 13.8, and 41.4. While the majority of Argentine surface supplies do not exceed 50 p.p.m., many ground waters exceed 300 p.p.m. While 200 p.p.m. may perhaps be regarded as the limit for desirability, it is yet a fact that many towns are drinking water of 900 p.p.m., or even more, of hardness, with no apparent detriment to health. Severe domestic inconveniences ensue when 250 p.p.m. is exceeded. *Chloride* is chiefly an index of distance from the ocean: an increase above the normal figure may denote pollution. *Sulfate* is a normal constituent usually present only in low concentration, but occasionally much higher. *Free carbon dioxide* is of significance only in connection with corrosion; it becomes a danger under conditions where it promotes the solution of lead. *Iodine* is a normal constituent, concentrations of 10 p.p.billion being desirable as prophylactic against goiter; when less is present, iodine tablets are being furnished for children from 6 to 10 years old. It has been found that when the proportion of sodium oxide to the combined oxides of calcium and magnesium present in the water is less than unity, iodine will generally be absent. The measure of organic matter known as *oxygen consumed* gives low values for ground waters and unpolluted shallow wells; but often quite high figures for natural surface waters: the test is chiefly of use as a measure of the effectiveness of purification processes. *Albumenoid ammonia* should not exceed 0.100 p.p.m. *Free ammonia* is often of vegetable origin and

without hygienic significance: should however as much as 0.500 p.p.m. be present, and also *nitrite*, then recent pollution is to be suspected, and if chloride concentration at same time be abnormal, then pollution may be regarded as certain. Presence of nitrite does not invariably signify pollution, even in surface water; but in conjunction with *ammonia* and/or *nitrate*, is always suspicious. Surface waters should be nitrite-free. Nitrate of mineral origin is without significance; if of organic origin, nitrite and/or ammonia will usually be present also. *Color, odor, taste,* and *turbidity* are usually without sanitary significance, but are always esthetically undesirable. While presence of sewage will influence unfavorably physical conditions, yet meteorological or geological phenomena are very much more often responsible. Turbidity in ground water is undesirable, as indicating defective underground filtration. *Iron* may be present in ferrous or ferric condition; it may be organically combined; more than 0.3 p.p.m. is undesirable. *Arsenic*: as shown on map, only 15.8% of Argentine supplies are arsenic-free; 78.5% contain from traces up to 0.100 p.p.m.; and 5.7% from 0.100 to 0.300 p.p.m. Chronic arsenosis, or Ayerza's disease, first noted by Abel Ayerza at Bell-Ville, Cordoba, is of slow onset, the usual period being five or six years, after which small eruptions appear on the hands and on the soles of the feet, which darken, developing frequently into arsenical cancer, while in other cases liver or heart ailments supervene which are often fatal. The waters were found to contain from 0.3 to 1.0 p.p.m. of arsenic, besides vanadium. The soil of this district contains from traces up to 0.4 p.p.m. of arsenic, concentration increasing with depth. Small traces of arsenic are not injurious: chronic arsenosis only appears after prolonged use of water containing about 0.3 p.p.m. Water with anything over 0.1 p.p.m. should, however, be rejected, and especially in presence of another toxic constituent such as vanadium. There is no doubt that the limit is variable, toxicity depending on other factors such as diet. *Vanadium*, as shown on map, occurs but rarely in Argentine waters, 94% of which are negative, 3% contain traces not exceeding 0.1 p.p.m., and 3%, from 0.1 to 0.5 p.p.m. Vanadium was first detected in Argentine water in '16, as result of abnormal color appearing in nitrate test. Biochemically, it is closely allied to arsenic, and when both are present together, a cumulative effect is probable. In minute traces, vanadium appears to have a therapeutic action, but in anything more than that, it is toxic: it is more toxic as vanadate than as vanadyl chloride: it is non-cumulative and quickly eliminated. Sodium metavanadate administered orally is tolerated in doses up to 12 mg. per kg. of animal (guinea-pig); 380 mg./kg. is fatal in 24–60 hours; hypodermically, 25 mg./kg. is tolerated, but 48 mg./kg. is fatal within 24 hours. Orally administered, it acts upon lungs, liver, stomach, intestine, spleen, and kidneys. A tentative limit of 0.1 p.p.m. is advocated, to be reduced in presence of other toxic constituents, such as arsenic. *Fluorine:* map of Argentina supplies is given showing that 72.4% contain less than 1 p.p.m.; 21%, from 1 to 2 p.p.m.; 5.3% from 2 to 5 p.p.m.; and 1.3%, over 5 p.p.m. Damon, in '29, first drew attention to the prevalence of dental fluorosis in Argentina. A tentative limit of 1 p.p.m. is advocated; it has to be remembered that water is not by any means the only source of ingested fluorine. Its removal from water is being investigated. Spectrographic analysis has revealed the presence in the Rio de La Plata not

only of fluorine, but also of manganese, strontium, zinc, lithium, titanium, and cerium. Brief mention is made of *boron*, *selenium*, *barium*, *chromium*, and *bromine*. *Lead* is found after contact with that metal of waters capable of dissolving it. The permissible limit is 0.1 p.p.m. It should always be sought for after coagulant-treated water has come in contact with lead. *Copper* is not thought to present any danger to health; permissible limit is set at 0.2 p.p.m. *Aluminum* presents no danger. *Zinc* is not often present, and never in dangerous concentration; permissible limit is set at 5 p.p.m. *Silver* might be found after oligodynamic purification, but only in negligible concentration. Residual *chlorine* is not yet called for in Argentina, nor is it considered to constitute a danger. *Micro-organisms:* (1) count on agar at 37°C. should not exceed 100; (2) most probable number of 2.2 per 100 cc. for coliform group should not be exceeded in surface waters, and ground water should be free; (3) water containing fecal *Esch. coli* should always be rejected.—*Frank Hannan.*

The Influence of Diet on the Chronic Toxicity of Selenium. M. I. SMITH. U. S. Pub. Hlth. Rpts. **54:** 1441 (Aug. 4, '39). Experiments on rats show that the toxicity of naturally occurring food selenium (Se) is largely determined by dietary factors. A level of intake of Se which is highly toxic and tissue-damaging when fed in a diet of low protein and high carbohydrate content is only slightly harmful, if at all, when fed in a diet of high protein and low carbohydrate content. The same level of Se intake in a low protein and high fat diet causes stunted growth and extensive loss of hair, but no other demonstrable tissue damage except some fatty degeneration of the polygonal cells of the liver. Such animals are quickly restored to a normal condition when returned to a balanced non-seleniferous diet. All available evidence indicates effects and toxicity of naturally occurring food Se are determined within certain limits not so much by the level of intake as hitherto supposed, as by the protein-selenium ratio in the diet. At a level of 10 p.p.m. of Se, a ratio of 1% protein in the diet to about 30 micrograms or less of Se per 100 grams of diet is of little, if any, toxicity. A ratio of 1% protein to 100 micrograms of Se per 100 grams of diet under the same conditions is dangerously toxic. Whether the quality as well as the quantity of protein in the diet plays a role in determining the toxicity of Se is not known at present. Article includes 3 charts and 4 figures.—*Ralph E. Noble.*

Note: This research points to the possibility that hitherto obscure cases of lead poisoning may also be the result of unbalanced diet. Note also the final sentence in the observations on arsenical toxicity in the abstract preceding. The irregular incidence of ill effects from lead in water, for example, may be largely governed by abnormalities in the diet of the person affected. It would seem that need of research regarding lead-dietary combinations is clearly indicated.—*H. E. Jordan.*

Cyanide Compounds in Water and Sewage. MARTIN STRELL. Gesundheits-Ing. **62:** 546 (Sep. 9, '39). The sources of cyanides in water are enumerated and the properties of the different compounds are discussed, giving chemical composition, formation and reactions. $(CN)_2$, HCN and the cyanides of

alkali and earth alkali metals are very poisonous. The lethal dose of KCN is given as 0.06 grams and therefore occasional temporary doses of 10 p.p.m. KCN in drinking water might be admissible, although a constant pollution should be limited to 0.5 p.p.m. The complex double salts like potassium ferro cyanide and thiocyanates are much less poisonous but should be kept to dilutions of less than 1:700. Compounds like Prussian blue are non-poisonous. Qualitative and quantitative tests for cyanides are given, based on the formation of Prussian blue or of silver cyanide. These tests do not have a very high degree of accuracy. Cyanides in water can be made relatively harmless by forming the complex double salts or they can be removed by precipitation as Prussian blue or by aeration after acidification.—*Max Suter.*

Salt-Taste Threshold in Man. CURT P. RICHTER AND ALICE MACLEAN. Am. J. Physiol. **126:** 1 ('39). Method for measuring salt-taste thresholds in man is presented. Av. concentration at which 53 adults first recognized difference in taste between salt solution and distilled water was 0.016% with median of 0.010%. Av. concentration at which they first recognized taste of salt was 0.087% with median of 0.065%.—*C. A.*

Danish Standard. Water Analysis—Bacteriological Examination. ≡ DS ≡. 265. Dansk Standardiseringsrad. Prescribed tests include: meat-peptone-gelatine count at 21°; meat-peptone-agar count at 37°; detection of coli-aerogenes group; determination of numbers in which they are present; and identification of *Esch. coli.* The coli-aerogenes group are defined as non-spore-forming, Gram-negative rods, fermenting lactose at 37° with gas evolution and acid formation, and in the test for them five successive ten-fold dilutions are employed and five tubes are taken of each dilution. A table is given by means of which, from the results of this test, the probable number of coli-aerogenes present is read off. For the *Esch. coli* identification the prescribed tests are: (1) either eosin-methylene-blue-agar, or Endo-agar; (2) Gram; (3) fermentation and acid formation in dextrose-peptone at 43°–45°; (4) indole; (5) methyl-red; (6) Voges-Proskauer; and (7) citrate. Unless both (4) and (5) are positive, *Esch. coli* is considered to be absent; with however this qualification that if the methyl-red test be faint, or in the rare event of a methyl-red +, indole −, the citrate test will then be regarded as decisive. Fermentation at 43°–45° as in (3) is conclusive as to presence of warm-blooded excremental material.—*Frank Hannan.*

INDUSTRIAL WATER SUPPLY

Industrial Uses of Water. W. D. COLLINS. J.N.E.W.W.A. **52:** 396 (Dec. '38). Author quotes from books on tea and coffee that very hard or very alkaline waters exert an unfavorable influence on the character of beverage coffee, whereas ordinary soft waters have little notable difference on quality; tea dealers recognize difference in water by supplying different blends for communities in various sections of the country. Soap and cleanser distributors interested in water hardness of communities in which their product sold. *Hardness:* Insoluble soaps formed in domestic and industrial washing due to hardness and process generally benefited by softening. In brewing 200–300 p.p.m. hardness considered beneficial, and in bread making it has been found

profitable to add nearly 2000 p.p.m. calcium sulfate. Calcium sulfate also used in soft waters containing sodium bicarbonate to prevent harm to soil when used for irrigation purposes. *Alkalinity:* Alkalinity may have influence on usefulness, sometimes total alkalinity being controlling factor, other times pH. Excessive alkalinity may be objectionable in brewing. In mfg. of hard candy desirable to have pH up to about 7.5 to prevent inversion of the sucrose which would make candy sticky when exposed to moist air; in the mfg. of fondant a low pH desirable to favor inversion and make product smooth. Alkalinity of beverage water reported to have striking effect on keeping quality, in one expt. beverage made with water using alk. similar to Miss. Valley supplies had, at end of 10 days, 45,000 times as many bacteria as with same beverage made with distilled water. Embalming formaldehyde solutions have been found to vary in preservative power with alkalinity of the water used. *Corrosiveness:* Differences of opinion exist as to extent practicable and profitable to treat waters to make them non-corrosive. Copper in solution will stain clothes and fixtures in laundry. Present day mfg. methods and standards are making scientific correction and treatment of water increasingly important.—*Martin E. Flentje.*

Water Quality Requirements for New England Industries. HERVEY J. SKINNER. J.N.E.W.W.A. **52:** 403 (Dec. '38). In New England paper and textile industries largest users of water where quality is important; other industries are laundries, pulp and paper mills. Scouring and bleaching of woolen goods requires 2000 gal. water for every 100 lbs. woolen goods, cotton goods 3000 to 4000 gal. and linen 10,000 gal. per 100 lbs. of product. Qualities of water which determine suitability for paper and textile mfg. use are: color, suspended matter, iron, manganese and hardness. In one case in mill making a special fiber product, spots occurring annually in Aug., were found due to bacterial growths which at other times of yr. were inhibited by cooler water temp. For high grade paper or textiles color should not be over 5. Fe and Mn may cause color trouble, usually Fe should be less than 0.10 p.p.m. and Mn as near 0 as possible. In paper industry hard water troublesome in sizing and coloring operations; in textile industry soft water essential in many operations. In processing cotton, material first boiled with NaOH to remove waxy materials, these with the NaOH form soluble soaps which precipitate with Ca and Mg present in water, affecting finished cloth. Hard water also objectionable in woolens and silks. In dyeing operations one of first prerequisites is a clean fiber, so must be free of impurities and above all insoluble soaps. Some dyestuffs used react directly with Ca and Mg of hard water causing streaks and blotches in dyed product. Description of industrial methods of purification given and importance of technical control cited. In discussion brought out that water consumption for wool scouring sometimes given as 20 gal. per lb. of wool, reported however 2 New Eng. plants using as low as 1.6 and 0.8 gal. per lb. of wool.—*Martin E. Flentje.*

The Treatment of Water for Industrial and Boiler Feed Uses. SHEPPARD T. POWELL. J.N.E.W.W.A. **52:** 418 (Dec. '38). Management of public water supplies has a definite, if limited, responsibility to industrial consumers, which is often given only minor consideration. Temp. and turbidity of supply

are primary characteristics for cooling and condensing water. Certain amount of turbidity, suspended matter and micro-organisms can be tolerated in cooling water, but excessive amounts cannot be present without imposing financial loss resulting from poor heat transfer in cooling equipment and high operation and maintenance charges. Examples given, with drawings, of efficient industrial water treatment basins using circular steel tanks. Author predicts tanks using sludge-blanket principle will have an increasing use in the treatment of industrial waters. Industrial processes often require limitation of silica content, for this reason filters often contain anthracite coal, calcite and magnetite instead of sand. Line drawings given with descriptions of several industrial water softening plants with unusual features. Under certain steam boiler operating conditions relatively small amounts of silica present in the boiler feed water will deposit silica compounds in the boiler forming dense, hard scale which is not controllable, within the boiler, by methods so far available. Proper operation and design of plant can reduce silica content to 3 p.p.m. Iron salts found most effective means of reduction. Protection of boilers against corrosion can ordinarily be accomplished by complete deaeration and control of pH of treated water.—*Martin E. Flentje.*

FLUORIDES IN WATER

Domestic Water and Dental Caries, Including Certain Epidemiological Aspects of Oral L. Acidophilus. H. Trendley Dean, Philip Jay, Francis A. Arnold, Jr., Frank J. McClure and Elias Elvove. U. S. Pub. Hlth. Repts. **54:** 862 (May 26, '39). Two Illinois cities (Galesburg and Monmouth), using a domestic water closely similar in source and mineral composition, show similarly low dental caries rates, 201 and 205 carious permanent teeth per 100 children, respectively. Two nearby cities (Macomb and Quincy), using a domestic water dissimilar in type and mineral composition from that of Galesburg and Monmouth, are characterized by dental caries rates double and treble those observed at Galesburg and Monmouth. The latter water supplies contain 1.8 and 1.7 p.p.m. of fluoride (F), the Macomb and Quincy waters only 0.2. While it seems reasonable to associate the *low* dental caries rates with the *higher* F content of the communal water supplies, the possibility that the composition of the domestic waters, other than the F content, may be a factor should not be overlooked. Using the approximal surfaces of the 4 superior incisors as a basis of measurement, there was 16 times as much interproximal caries in Macomb and Quincy as in Galesburg and Monmouth. The amount of *L. acidophilus* in the saliva closely reflected the difference in the dental caries rates between Galesburg and Quincy. Bacteriological studies were not made at Monmouth and Macomb mainly because of the smaller number of children available for study. The quantity of amylase secreted in the saliva disclosed no group population differences between Galesburg and Quincy. From an epidemiological standpoint, the author concludes it is difficult to ascribe these differences to any cause other than the common water supply. Article includes 13 tables, 4 figures and 16 references.—*Ralph E. Noble.*

Editorial Note: *Caries* is ulceration or decay of bone structure. *Dental caries* is the medical term ordinarily applied to the common decay or cavitation of the teeth. The lay reader of the foregoing abstract and other similar dis-

cussions must be careful to distinguish between *dental caries* and *mottled enamel*. The latter is the term applied to the particular dental defect which is generally attributed to an excess of fluorides in the food or water. For a more complete discussion of mottled enamel see Journal A.W.W.A. **23**: 214 (Feb. '31).

Possible Relationship between the Fluorine Content of Enamel and Resistance to Dental Caries. W. D. ARMSTRONG AND P. J. BREKHUS. J. Dental Res. **17**: 393 ('38). The suggestion that an increased resistance to dental caries may be due to a certain amount of fluorine in the composition of the enamel, is not a new one, but Armstrong and Brekhus have now brought the subject within demonstrable bounds of possibility. The observations made from time to time on mottled enamel and the fact that caries incidence in such teeth is low, have attracted attention in the popular press and have stimulated the present authors to conduct analyses of sound and carious teeth with a view of discovering the fluorine content. Tabular data demonstrate the protective effect of fluorine.—*B. H.*

Fluorides and Mottled Enamel in Illinois. CHARLES F. DEATHERAGE, CLARENCE W. KLASSEN AND JAMES C. WEART. Ill. Dental J. **8**: 194 ('39). The concn. of F in the H_2O supplies of Illinois is reported. Greatest concn. is found in northern part of the state, and whenever more than 1 p.p.m. is found, mottled enamel occurs. There is some evidence that less dental caries occurs in the regions rich in F.—*C. A.*

Some Characteristics of Endemic Dental Fluorosis (Mottled Enamel). H. T. DEAN. Southwest W. W. J. **20**: 9: 11 (Dec. '38). There are already 375 known endemic areas in the United States, divided among 26 states. The West Texas-Panhandle region constitutes the largest affected territory in this country. Of the many and widely separated foreign countries, the Argentine Republic is most seriously affected. When the mean annual fluoride (F) content is 1 p.p.m. about 12% of the children are affected. When 2 p.p.m., about 50% are affected. Children should not use water containing over 1 p.p.m. of fluoride during the susceptible period which is from 3 months to 8-10 years of age. D. emphasizes the unusually long time of 8 to 10 years before the effect of fluoride in the water is shown by the eruption of the permanent teeth and the need for exact knowledge of the fluoride content of new or projected supplies. Since there appears to be a limited immunity from dental caries among children residing in endemic areas, the possibility of controlling dental caries through a domestic water supply warrants a thorough epidemiological study. Twenty-six references given.—*O. M. Smith.*

Occurrence, Pathological Aspects, and Treatment of Fluoride Waters. M. STARR NICHOLS. Am. J. Pub. Health **29**: 991 (Sep. '39). Conc. of fluorides in drinking water must be below 1 p.p.m. not to cause mottled enamel, such water must be obtained by selection of source low in fluorides or through removal of fluorides by treatment. Origin of all fluorine bearing minerals is igneous rocks underlying sedimentary formations. Fluorine as fluorosilicates, sodium fluoride, rock phosphate, and calcium phosphate show toxicity to the

animal body in the order given and in comparatively small amounts. As little as 4 grams of fluorides has proven fatal to humans when taken by stomach. Expts. on removal have included treatment with alum (10 gr. per gal.), activated alumina, reduction by lime softening, through aid of tricalcium phosphate, magnesium oxide or magnesium hydroxide. Use of tricalcium phosphate in filter bed with regeneration of bed with 1% sodium hydroxide and neutralization with dilute acid (HCl, H_2CO_3 patented) described. Problem not entirely solved.—*Martin E. Flentje.*

On Fluorosis from Drinking Water or the "Mottled Enamel" of Americans. V. MÜHLENBACH. Kleine Mitt. Ver Wasser-, Boden- u. Lufthyg. **14:** 76 ('38). A summary of literature dealing with the occurrence, cause, symptoms, and prevention of dental fluorosis, with 95 references. Water was early recognized as a cause of the complaint but it was not until '31 that fluorine was realized to be the dangerous constituent. Dental fluorosis may occur where the water supply contains over about 1 mg. per liter of fluorine. In America numerous surveys have been made; in '36 there were 335 endemic areas in 25 states, 86% of which lie west of the Mississippi. Cases of dental fluorosis have been found in almost every part of the world. The damaged teeth cannot be treated in any way and it is therefore necessary either to obtain a different water supply free from fluorine or to treat the water to reduce the content of fluorine below 1 mg. per liter. Literature on the determination of the content of fluorine and on methods for its removal from water is discussed.—*W. P. R.*

Studies on the Removal of Fluorine from Drinking Waters in the State of Iowa. C. A. KEMPF, ET AL. Iowa Acad. Sci. Proc. **43:** 191 ('36); Expt. Sta. Record, U. S. Dept. of Agri. **80:** 2: 257 (Feb. '39). In continuation of previous studies (E.S.R., **69:** 620) the authors give a preliminary report on a large-scale method for the removal of fluorine from water which consists of a 25-hr. treatment with aluminum sulfate, with the pH of the water below 7. The method was tested on Ankeny city water and the fluoride concentration was lowered from 7 to 10 p.p.m. to from 1.5 to 2 p.p.m. of water.—*P. H. E. A.*

Results of Recent Mottled Enamel Surveys in Ohio. D. L. HOUSER AND W. H. KNOX. Ann. Rept. Ohio Conf. Water Purif. **18:** 27 ('38). Surveys were conducted in 19 villages where fluorine content of water supply was considerable and at 6 where it was negligible. In former, 30–50% of pupils were affected; in latter, 20–30%. Latter result rather disconcerting. Case histories of affected pupils might explain some results which can now only be surmised. It is concluded that objectionable fluoride content may be 1.5 p.p.m. or lower. Well waters in Ohio which have very high hardness are likely to have high fluorine content; those with moderate or even low hardness may also have objectionable fluorine content.—*R. E. Thompson.*

Fluoride in the Ground Water of Avoyelles and Rapides Parishes, Louisiana. JOHN C. MAHER. U. S. Geol. Surv. Geol. Pamphlet No. 1, Dept. of Conserv., La. Geol. Surv. (Jun. '39). In connection with ground water investigations

in Louisiana a study has been made of the occurrence of fluoride in ground water supplies in Avoyelles and Rapides parishes (counties). Water supplies in this area are obtained from wells penetrating the Miocene and younger aquifers which range in depth from 15′ to over 1200′. Water from 45 of 48 wells penetrating the Miocene from which samples were collected contained fluoride, 19 with a Fl content of over 1 p.p.m. Water from the older and coarser sediments beneath the Miocene was free from fluoride. Some of the samples from younger strata overlying the Miocene contained fluoride but not in excess of 0.2 p.p.m.—*J. H. O'Neill.*

Investigation of the Occurrence of Fluorine in Drinking Water in New Mexico and the Danger of Fluorine to Health. J. D. CLARK AND E. H. MANN. Bull. Univ. N. Mex. Chem. Ser. No. 329 ('38); Chem. Zbl. **1:** 3605 ('39). Fifty per cent of samples of drinking water from the state of New Mexico contained over 0.9 mg. of fluorine per liter. The concentration of fluorine can be reduced by boiling the water with bone ash.—*W. P. R.*

The Presence of Fluorine in Waters of the Netherlands. M. E. STAS, L. H. LOUWE KOOIJMANS AND J. J. VAN IJSSEL. Water **21:** 1: 28 ('37); Chem. Zentr. **II:** 641 ('37). A review is given of investigations of attacks on the dental enamel caused by F in drinking water. Analyses are reported of the F content of samples from 132 pumping stations and 329 private supplies. In only one case was the F content greater than the limiting safe value of 0.9 p.p.m. In this one exception the higher F content was explained by geol. conditions.—*C. A.*

Mottled Enamel Among the Inhabitants of Krawang. OUW ENG LIANG. Meded. Dienst d. Volksgezondheid in Nederl.-Indië **28:** 1 ('39). From an extensive investigation of several hundreds of persons, children and adults, in eight village communities of the Segalaherang district of the Krawang regency, the author found a considerable proportion ★[the actual figures are not given]★ with mottled enamel. Examination of the water supply, which is the River Tjipabelah arising in the northern slopes of the volcano Tangkoeban Praoe, showed the fluorine content to be as high as 3.2 mg. per liter near the source, less downstream. The dental condition is well shown in photographs.—*B. H.*

Does Any Relationship Exist Between the Content of Fluorine in Drinking Water and Goiter? TH. VON FELLENBERG. Mitt. Lebensm. Hyg. **29:** 276 ('38). The problem was studied on the basis of 64 samples of drinking water, taken from places in which endemic goiter was known to prevail. It was established that where this endemia was low the water was always low in F; but strong endemia can occur both in places where much F is present in water and also in those where little F is found. A final conclusion can be drawn only after the drinking water as well as the food has been examd. for the content of F. The method of F. detn., especially in water, developed by the author previously was revised and it is described in detail.—*C. A.*

BACTERIOLOGY

Note on the Successive Dilution Method for Estimating Bacterial Populations. R. D. Gordon and C. E. ZoBell. Zent. f. Bakt. (Ger.) **99:** 318 ('38). In a previous paper Gordon criticized on theoretical grounds the use by Halvorson and Ziegler of the modal value in estimating the probable density of a bacterial population by the dilution method. In the present paper experimental evidence is brought to confirm this theoretical conclusion. According to Halvorson and Ziegler, if ten tubes are inoculated with each of three successive dilutions, and p1, p2, and p3 represent the number of tubes showing growth in the lowest, middle, and highest dilutions respectively, then the propability of this (compound) event can be computed in terms of the actual mean density of bacteria in the original material, and the Bayes-Laplace formula ("inverse probability") can then be used to determine the probability that this event is the result of any particular value of the density, assuming that the organisms are homogeneously distributed throughout the original suspension. Halvorson and Ziegler take the *mode* of this probability as an estimate of the actual density. This value of the density is the one having the greatest individual probability. In the present work comparative counts were made by the plate count and the dilution method. Considerable disagreement was found between the two counts when Halvorson and Ziegler's formula was used for estimating the bacterial density by the dilution method. Such disagreement is believed to be due to variations in the skewness of the posterior ("inverse") distributions. Evidence is brought to show that if the *geometric means* are taken instead of the modal values, the results of the plate count and dilution methods are in closer agreement. It is proposed to work out tables with a complete set of geometric mean values.—*B. H.*

Productivity of Media Containing Milk for Recently Isolated Strains of the Coliform Group. M. T. Bartram and L. A. Black. J. Bact. **37:** 371 (Apr. '39). Two strains each of *Escherichia, Aerobacter,* and intermediates recently isolated from raw milk were inoculated into five liquid and seven solid media, in addition to standard lactose broth and standard methods agar, as controls. A modification of the Butterfield method was followed with two dilutions inoculated into 10 tubes of each medium. Two dilutions were plated in triplicate in the solid media and 1 ml. of sterile whole milk added to each plate or tube before the organisms. The productivity of the liquid media tested, based on Hoskins' Most Probable Numbers was, in order of descending sensitivity: (1) methylene-blue brom-cresol-purple, (2) fuchsin lactose, (3) brilliant-green bile, (4) formate-ricinoleate, (5) Eijkman medium. None were as productive as standard lactose. The solid media ranked as follows: (1) neutral-red bile, (2) violet-red bile, (3) Endo, (4) brilliant-green lactose bile, (5) lactose taurocholate, (6) desoxycholate, and (7) trypaflavine agar. The unsatisfactory results obtained with desoxycholate agar may be accounted for by its low productivity with the cultures of the intermediate group used. Neutral-red bile and violet-red bile gave counts considerably higher than those obtained on standard agar plus milk. ★[May it be added that comparative tests (J.A.W.W.A. **27:** 1143 (Sep. '35)) have suggested the likelihood that

culture media tested for productivity with pure strains will show a different order of sensitivity when tested with raw waters or used under practical conditions. McCrady's report (Am. J. Pub. Health **27**: 1243 (Dec. '37), abstracted in J.A.W.W.A. **30**: 1421 (Aug. '38)) of cooperative work with different waters and different media, including some used by the authors, shows another order of sensitivity and tends to bear out the above comment.]★—*Ralph E. Noble.*

Yellow Typhoid Bacilli. EDITORIAL. J. Am. Med. Assoc. **112**: 1346 (Apr. 8, '39). The transformation in the test tube of nonpathogenic flavobacteria into virulent typhoid bacilli, recently alleged by Dresel and his co-workers of Leipzig, Germany, (Dresel, E. G., and Graf, Willi: Arch. f. Hyg. u. Bakt. **119**: 3: 153 ('37). Dresel, E. G., and Lotsch, Herbert, *ibid.* **120**: 4 and 5: 286 ('38)) challenges conventional theories of typhoid epidemiology. It is reported that by repeated "single cell" cultivation of several strains of flavobacteria isolated from urine, nonpigmented variants were eventually obtained morphologically, culturally, and antigenically identical with standard strains of *B. typhosus*. Reversing the process, the investigators were able to dissociate from a standard laboratory strain of *B. typhosus* a yellow variant (*Bacterium typhi flavum*). More than sixty species of flavobacteria are recognized by American bacteriologists, most of them regarded as common saprophytes in surface waters. Medical bacteriologists of the past have usually disregarded these yellow bacteria as of little clinical interest, since none of the known yellow strains had been shown to be pathogenic for man or lower animals. The conclusions of the Leipzig bacteriologists, however, that some of the species of flavobacteria are merely disguised forms of *Bacillus typhosus* may lead to a critical reexamination of the experimental evidence leading to this conclusion.—*J. H. O'Neill.*

Incidence and Characters of Vibrios in Waters in Northern India. J. TAYLOR AND M. L. AHUJA. Indian Jour. Med. Research **26**: 1: 1 (Jul. '38). The presence of certain vibrios in water had been suspected of playing a very small part in the aetiology of cholera. In confirming this finding, the authors found "that vibrios are almost universally present (90 out of 91 samples) in unprotected wells, tanks, and rivers of Northern India where cholera is not endemic, and that their presence is in no way related to contamination from cholera sources. From individual sources vibrios of several different types may be isolated and the vibrio group in water is extremely heterogeneous. . . . It is obvious that with this almost universal distribution of vibrios in water, including water which may be used for drinking purposes, vibrios must frequently obtain entrance to the intestinal tract of those using the water and the appearance of such vibrios in stools of healthy individuals or in cholera cases is to be expected." The biochemical and serological reactions of these apparently harmless vibrios is described.—*P. H. E. A.*

A New and Simple Method of Flagella Staining and a Contribution to the Question of Differentiating Coli and Aerogenes Bacteria. Kleine Mitteilungen. **15**: 71 (Jan.–May '39). Anatomically, biologically and biochemically the con-

cepts of *Esch. coli* and *Aer. aerogenes* are related to groups of organisms which are difficult to be differentiated. Koser's citrate media as well as the methyl red test and the Voges Proskauer reaction do not always give clear results. On the other hand the motility of *Esch. coli* is always charaeteristic compared to the always immotile *Aer. aerogenes*. Observation in the hanging drop is not satisfactory as the motility of *Esch. coli* is often small. A new method for staining the flagella is given in detail based on a mordant containing tannin and staining with silver nitrate and ammonia. Careful cleaning of the slide is required and few organisms widely separated should be present on the slide. If flagella are present we have *Esch. coli*. However a clear separation of *Esch. coli* and *Aer. aerogenes* is not possible even with this method as strains of *Esch. coli* were found that did not have flagella.—*Max Suter.*

A Faster Method for Bacteriological Analysis of Water with Membrane Filters. E. M. Nisnevich and M. M. Rozenblat. Lab. Prakt. (U. S. S. R.) **1:** 13 ('39). Carefully washed used celluloid x-ray plates can be successfully used as filters. The filters are prepared by taking 2.5–3 grams of the x-ray plate, 50 cc. acetone and 25 cc. wine alc. The mixt. is shaken, filtered through a cotton filter and is then poured over a smooth 30 x 30 cm. glass plate. After drying the filter, it is wetted with water, taken off the glass surface and again dried between 2 sheets of filter plates. They are inserted into Zeiss' filter and sterilized for 1–1.5 hrs. at 130°. A sheet of filter paper of the same size is placed on the metal plate of the Zeiss' filter to give the filters the necessary strength. Four hundred expts. performed with artificially prepd. mixts. contg. *Esch. coli, Esch. communior* and *Aer. aerogenes* in water gave consistent results in all cases after 16–20 hrs. The time required for investigations is reduced to a max. of 24 hrs. giving as accurate results as those obtained by the standard method requiring much longer time.—*C. A.*

Filtration of Water as a Method of Concentration in Testing for Pathogenic Bacteria. F. D. Monica. Riv. sanit. sicil. **26:** 791 ('38); Bull. off. int. Hyg. publ. **31:** 352 ('39). Describes an adaptation of Weinberg's method for detecting the presence of pathogenic bacteria in water. Apparatus consists of an open tube 20 cm. long and 5 mm. diam., narrowing 5 cm. from its lower end. A pad of adsorbent cottonwool is put down the tube to the point where it narrows; on this is placed about 1 gram of compressed kieselguhr and another pad of cottonwool and the whole is sterilized in an autoclave. Has been used to filter quantities of from 500 cc. to 2 liters of liquid containing in all not more than 4–5 bacteria. By adding a nutrient medium and incubating, positive results were always obtained with liquids containing *B. typhosus, Vibrio cholerae,* or *Esch. coli.*—*W. P. R.*

Growths in Distribution Systems Especially in Connection with the Appearance of Iron Bacteria. L. Kooijmans. Water (Neth.) **21:** 76 ('37); Gesundheits.-Ing. **61:** 308 ('38). Development of iron bacteria can be controlled by reducing the content of organic matter in water and by treatment with lime. More important, however, to remove iron from the water and to prevent iron

being taken up by the water as it passes through the pipes. Deposits in pipes can be rendered harmless by continuous chlorination or ammonia-chlorine treatment. Wells can be disinfected with copper sulfate. Water in collecting tanks can be treated with excess chlorine at the inlet and dechlorinated at the outlet. Aftergrowth in the distribution system may result in a high bacterial count and even the presence of *Esch. coli* in a water originally of a high degree of purity. This phenomenon appears in chlorinated water when the chlorine has disappeared. It has been suggested that chlorine in a very dilute solution encourages growth of organisms, that protozoa, which are very sensitive to chlorine, are prevented from destroying bacteria, and that organic matter is broken down by chlorine to lower organic compounds which form a nutrient medium for chlorine-resistant spores. In America the disinfection of new water mains is being widely adopted. The result is not always satisfactory. It may be that mechanical cleaning is neglected and sand grains, etc. in the pipes afford a certain amount of protection to bacteria.—*W. P. R.*

Comparative Resistance of Recently Isolated and Older Laboratory Strains of E. Typhosa to Action of Chloramine. P. KABLER, G. O. PIERCE, AND G. S. MICHAELSEN. J. Bact. **37:** 1 ('39). Experiments described which show that different strains of *B. typhosus* isolated at approx. same time may vary considerably in resistance to action of water treated by the chloramine process to give a residual chlorine content of 0.20 p.p.m.; the resistance of the organisms is materially reduced by prolonged growth on artificial media. Shortly after isolation, strains of *B. typhosus* were killed in 2.18 hrs. in concentrations of 0.2 p.p.m. of chlorine; after they had been grown on culture media for one year the organisms were killed in 0.9 hrs. At temperatures of 1–3°C. the time required to kill the organisms is 3–5 times greater than that at 24.5–26°C. There is no correlation between the time required to kill the bacteria and the pH value of the water or the concentration of dissolved oxygen, iron or manganese.—*W. P. R.*

Studies on the Influence of Chlorination on the Sensitivity to Temperature of Esch. coli. G. NORDGREN. Nord. hyg. Tidskr. **19:** 209 ('38), (German summary 222); Zbl. ges. Hyg. **43:** 103 ('39). Author attributes failure of the Eijkman test to indicate the presence of *Esch. coli* in polluted water from swimming pools to the effect of chlorine on the organism. In 9 out of 10 experiments, *Esch. coli* taken from chlorinated water had lost ability to grow on and ferment glucose broth at 46°C. Optimum temperatures for growth of 4 strains fell from 42–43° to 37°–40°C. after chlorination of the water. The Eijkman test, i.e. the production of acid and gas in glucose broth inoculated with the sample and incubated at 46°C., is therefore unreliable for the examination of chlorinated water; the sensitivity of *Esch. coli* towards chlorine can vary and a decrease in the optimum temperature for growth of *Esch. coli* is not likely to be accompanied by an increase in the toxic action of the chlorinated water on typhoid, paratyphoid and dysentery bacilli. The author recommends testing for gas production at 37°C. and plating on Endo agar and congo-agar. —*W. P. R.*

Determination of the Heat Resistance of Non-Spore-Forming Bacteria. Food Research (Champaign, Ill.) **4:** 21 ('39). Forty-eight hour cultures of *Esch. coli* on nutrient agar at 37°C. were washed off and suspended in 10 cc. of M/15 sodium potassium phosphate buffer solution; the density was adjusted in accordance with a haemocytometer count. Tubes of buffer solution were heated in a constant temperature bath to the required level and then inoculated with the requisite number of organisms. The tubes were not sealed. At the end of the heating period, the tubes were cooled; 0.5 cc. of concentrated broth was added to the suspension, and the tubes were incubated to see if growth occurred. Considerable variation was found in the end point of individual tubes. The most satisfactory method consisted in heating 25 tubes at each of four temperatures for different lengths of time, and plotting the logarithms of the percentage of tubes showing growth. This yielded a straight line curve which by extrapolation gave the sterility point with a fair approach to accuracy. The main burden of this paper is to show how unreliable conclusions are if they are drawn from observations on a few tubes exposed to widely different temperatures. Close spacing of temperatures is required, and heating periods should be fairly long. The thermal death time for a strain of *Esch. coli* was found to be between 20 and 30 min. at 57.3°C.—*B. H.*

Bacteriological Water Examination. T. Folpmers. Antonie van Leeuwenhoek (Neth.) **1:** No. 2 ('38); Water (Neth.) **23:** 136 (Jul. 14, '39). Study dealing with disappearance of fecal streptococci during treatment of water at Rotterdam. The Meuse water at the intake contains the greatest number of streptococci (also greatest number of coliforms) during high water, caused by sewage pollution. Soon a pumping station at the inlet channel will allow pumping at lowest tide. Through pre-sedimentation some streptococci disappear, by prefiltration again a certain percentage, but the largest number are removed by slow sand filtration, principally through the activity of protozoa. Results of experiments appear to prove this contention. For determination of fecal streptococci two solutions are recommended which show definite growth and inhibit other bacteria: (a) peptone poulenc 1%, lactose 1%, NaCl 0.5%, pH 7.0, incubation at 45°C. (b) peptone poulenc 1%, caffeine 1%, glucose 0.1%, Leibig extract 0.3%, pH 7.0, incubation in closed bottles at 37°C.—*Willem Rudolfs.*

Incubation at 44°C. as a Means of Determining the Presence of Esch. Coli. R. Ferramola and J. J. Monteverde. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 248 (Mar. '39). Work done by the authors with sewage samples verifies the results obtained in a previous study (See abstract J. A. W. W. A. **31:** 416 (Feb. '39)) in which use was made of MacConkey's broth at 44°C. incubation temperature for the rapid differentiation of *Esch. coli* from intermediates and other members of the coliform group. *Esch. coli* produced acid and gas while the other organisms did not modify the medium, or produced acid only, after 24 to 48 hrs. incubation. The experimental technique consisted in streaking two loopfuls of sample on eosine methylene blue agar. After 24 hrs. at 37°C., typical colonies were transferred to lactose broth. Another eosine methylene blue plate was streaked with material from each lactose broth tube showing gas after 24 hrs. at 37°C. From colonies develop-

ing within 24 hrs. at 37°C. on the second E. M. B. plate, transfers were made to MacConkey's broth, for the 44°C. incubation, and to the proper media for determining the M. R., V. P. Indol and Citrate reactions. Out of 400 colonies of various types studied, a correlation of 97.8% for *Esch. coli* fecal type and of 98.9% for *Aer. aerogenes, Aer. cloacae* and intermediate strains was obtained. The following distribution of coliform organisms was found in Buenos Aires sewage: *Esch. coli* type I, 56.00%; *Esch. coli* type II, 1.25%; intermediate type I, 12.25%; intermediate type II, 13.25%, *Aer. aerogenes* type I, 15.50%; *Aer. aerogenes* type II, 1.25%; *Aer. cloacae*, 0.50%.—*J. M. Sanchis.*

Experiences With a New Method for the Sanitary Examination of Water. I. RAUL FERRAMOLA, AUGUSTO A. GILARDON AND GUILLERMO SOSA. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 354 (Apr. '39). Need for a rapid, simple and accurate method for the bacteriological examination of water is emphasized and the weak points of the present British and American standard procedures are discussed. The promising results obtained by Bardsley in Manchester with a method suggested by Prof. Wilson of London University induced the authors to try this procedure for the examination of waters in Argentina. The method consists in transferring suitable amounts of sample to MacConkey's broth (prepared according to the British Public Health Ministry formula) and incubating at 37°C. for 48 hrs. The number of positive tubes (those showing acid and gas) allow the estimation of the M. P. N. of coliforms in the sample. From each positive MacConkey tube, transfers are made, with a straight needle, to Koser's citrate media and to a second MacConkey broth tube. The citrate tubes are incubated at 37°C. for 72 hrs., while the second MacConkey tubes are placed during 48 hrs. in a water bath the temperature of which is maintained at 44°C. (± 1°C.). The number of positive citrate tubes is used for estimating the concentration in the sample of intermediates, aerogenes, and cloacae organisms, while the positive MacConkey's at 44°C. give an indication of the number of *Esch. coli* fecal type present. To determine the reliability of the first step of this procedure, transfers were made, from the highest dilutions giving positive MacConkey's at 37°C., to E. M. B. plates. After incubating these plates at 37°C. during 24 hrs., typical colonies were transferred to standard lactose broth and agar slants in order to determine the lactose fermenting capacity of the organisms and their morphological and staining characteristics (American standard completed test). Of the samples collected in the City of Buenos Aires (which included raw, settled, settled and chlorinated, filtered, and filtered and chlorinated waters), all of the 24 hr. MacConkey positives were found to be coliforms by the completed test, while 94% of the 48 hr. positives gave typical findings by the completed American standard. Of the 24 hr. MacConkey positives given by waters obtained in the interior of the Republic, 96% of those from surface waters and 74% of those from ground waters were found to contain coliforms, while 77% of the surface water MacConkey positives at 48 hrs. and 73% of those from ground water, also positives at 48 hrs., were in agreement with the findings of the completed test. The authors believe that the ease with which coliform colonies may sometimes go unnoticed on E. M. B. plates, prevented a closer correlation between the MacConkey broth results and those of the completed test.

II. Raul Ferramola, Adolfo Gelstein, Vincente O. Dolcetti. *Ibid.* **3:** 584 (Jun. '39). The authors present laboratory data obtained in order to determine the reliability of the results given by the second step of Wilson's procedure for the bacteriological examination of water. From the highest dilutions giving a positive reaction within 48 hrs. in MacConkey's broth incubated at 44°C., transfers were made to E. M. B. plates. After 24 hrs. incubation at 37°, three typical colonies from each plate were inoculated separately into the usual media for the determination of the MR, VP, citrate, and indol reactions. The results indicated that 97% of the 24 hr. positive tubes from surface samples, 96% of surface water positives at 48 hrs., 95.5% of ground water positives at 24 hrs. and 93% of the ground water positives at 48 hrs., showed the presence of *Esch. coli* fecal type I or II by the differentiative reactions. To further verify its applicability, the Wilson procedure was used to examine over one thousand coliform strains isolated from Buenos Aires sewage and from the feces of animals usually found in watersheds. The satisfactory correlation found in all cases between the MacConkey broth results at 44°C. and those obtained by the tentative differentiative tests proposed by "Standard Methods" of the A. P. H. A. and A. W. W. A., indicates the possibility of ascertaining the presence of *Esch. coli* fecal type in a fraction of the time needed by the present British and American procedures. The results of similar experiments made to verify the presence of intermediates and aerogenes organisms in the positive citrate tubes, were not equally satisfactory since, even though there was a 92% correlation for the 24 hr. transplants, the correlation for the 48 hr. transfers was only 57% for surface waters and 64% for underground supplies. As a result of these studies, however, the authors believe Wilson's technique to be a valuable contribution towards the improvement of existing procedures for the bacteriological examination of water, and express their desire that the method be tried by bacteriologists elsewhere, especially in South America, so that definite conclusions in regard to its applicability and limitations may be drawn from their combined experiences. —*J. M. Sanchis.*

Note: Refer to "British Methods of Analysis" (J. A. W. W. A. **31:** 1226 (Jul. '39)) for discussion of use of MacConkey's broth. Note also that authors have overlooked the "liquid confirmation" method in the 8th Edition of Standard Methods.—*Ed.*

The Coli Aerogenes Index of Pollution Used in the Bacteriological Analysis of Water. T. N. S. Raghavachari and P. V. Seetharama Iyer. Ind. J. Med. Res. (Calcutta) **26:** 867 (Apr. '39). Literature relative to differentiation of fecal and non-fecal organisms of coliform group of bacteria is reviewed briefly. Wilson, et al (Br. Med. Res. Council, Special Rept. Series No. 206, '35) recommended MacConkey broth at 37°C. for presumptive coliform count, MacConkey broth at 44°C. for fecal coli count and citrate test for intermediate-aerogenes-cloacae count. Results obtained by Wilson were confirmed by Bardsley. Authors examined 12 samples of Indian waters to determine applicability of 44°C. test for differentiation of fecal and non-fecal types of organisms, obtaining 416 cultures, 186 being aerogenes-like organisms. From 60 to 70% of latter produced acid and gas in 24–48 hrs. in MacConkey broth at

44°C., temp. being accurately maintained in water bath. Only about 50% of cultures capable of growing at 44°C. were found to contain typical *Esch. coli*. Cultures sent to Wilson for confirmation of results behaved similarly. It is therefore concluded that MacConkey test at 44°C. can be considered sufficiently specific for differentiating coliform organisms into fecal and non-fecal types *only if it can be proved* that aerogenes-like organisms capable of growing at 44°C. are normal intestinal organisms in India. 24 references. —*R. E. Thompson.*

Longevity of Coliform Organisms in Water Stored Under Natural Conditions. T. N. S. RAGHAVACHARI AND P. V. SEETHARAMA IYER. Ind. J. Med. Res. (Calcutta) **26:** 877 (Apr. '39). Clemesha, in '08–'09 study at King Institute, with which authors are connected, divided coliform organisms found in feces and water into 3 classes on basis of ability to resist action of sunlight, and claimed that one or more resistant species will survive the others. This classification has been regularly applied at Institute since '08, together with, since '21, other classifications based on more recently devised differential tests. Experience of some 25 yrs. is not in accord with Clemesha's findings and practice has evolved of using numerical estimate of lactose fermenters as sole criterion in routine assay of water supplies. To determine effect of storage and sunlight, tests were made in 3 masonry tanks, 30′ x 30′ x 6′ (5.5′ effective depth), of about 31,000 gal. capac. each. Only possible source of contamination was bird droppings and aquatic life entering at time of filling. Tanks filled with Adyar R. water on Nov. 8, '37, and water allowed to remain therein until Feb. 28, '38, representing monsoon conditions, and again filled on Mar. 8, '38, and allowed to remain until Sep. 5, '38, representing hot weather conditions. Samples withdrawn and examined periodically. *B. vesiculosum*, which is present in feces, cow-dung and water with remarkable consistency and which under modern scheme of classification is member of the coliform group, was found by Clemesha to be resistant to sunlight and storage. Classification of coliform bacteria based on resistance to storage and sunlight would, therefore, appear to be of little or no value. In both series of tests, *Esch. coli* (Clemesha's class No. 1) was found to survive for considerable time and showed no appreciable reduction in nos. during at least first 4–5 weeks of exposure. In first series, ratio of *Esch. coli* to total coliform organisms was as high at end of period as at beginning of experiment. In second series, proportion of *Esch. coli* was lower from start of experiment and there was progressive reduction in numerical strength after first 6 weeks but they were still present after 4 months. Attention is drawn to fact that there is reason to believe that some aerogenes forms are truly fecal type. The only fairly reliable index for assay of potability is the coliform count.—*R. E. Thompson.*

LABORATORY METHODS AND EQUIPMENT

Enumeration of Living Organisms. H. W. JULIUS. Antonie van Leeuwenhoek Nederl. Tijdschr. v. Hyg. Microbiol. en Serol. **5:** 28 ('38). Plate methods of enumeration of living bacteria in water, milk, etc., are those commonly used in laboratories. If, however, large numbers of examinations have to be

made, for example 200 plates at a time, the strain on the resources, time and staff of the laboratory becomes unbearable. Methods have been introduced to obviate this: all have the disadvantages of being micromethods. Author has devised and introduced a method of roll culture determinations, which has succeeded admirably. Instead of test tubes, however, he uses small flasks which are obtainable commerically. They have a rubber cap and a bakelite screw top. Found that roll cultures of melted agar could be much better made when the revolutions were 600 to 800 instead of 100 per min. Apparatus constructed with a motor and a series of axles on which the small flasks were fitted, by which six or more roll cultures could be prepared simultaneously. Drop pipettes for delivery of a standard drop were made by means of a metal gage from capillary tubes with a central bulb and these were fitted into a rubber stopper with teat arrangement. Finally the small flasks after incubation and development of colonies were fitted on a spindle capable of rotating and examined with lens and mirror to count the colonies. With this arrangement there was no fear of going over any area more than once, and, as each field examined was very small, there was no difficulty in counting 500 to 1,000 colonies, whereas counting on an ordinary plate becomes almost impossible if the number is over 300. Much saving of time and energy has been effected by the procedure described. Four illustrations.—*B. H.*

Simplified Method for Making Photographic Records of Petri Dish Cultures without Camera. E. Maier. J. Lab. & Clin. Med. **23:** 1299 ('38). A method is described, similar to that used by previous workers, for making photographic records of plate cultures by direct printing on sensitized paper. A piece of sensitized paper (Azo paper of Eastman Kodak Co. is recommended) is placed over the Petri dish in a dark room and exposed to electric light from a 75–100 watt bulb situated about 1.5–2.0 meters distant for 10–25 seconds, the dish being between the paper and the light. The print is developed and fixed by the usual methods.—*B. H.*

Photographing Microscopic Organisms. Fred E. Smith. W. W. and Sew. **86:** 230 (Jun. '39). Description of inexpensive apparatus for making photographs through microscope accompanied by samples of results and procedural details.—*H. E. Hudson, Jr.*

Some Applications of the Polarizing Microscope to Water Conditioning Problems. Everett P. Partridge. Proc. A. S. T. M. **37:** Part II: 600 ('37). A general discussion is given of the uses of the polarizing microscope, with reference to the technique employed in examination of scale deposits and corroded metal surfaces, the application of chemical microscopy, and the identification of crystalline materials by measurement of optical properties. Definite identification has been made of $CaSO_4$, $CaCO_3$ (calcite and aragonite), $Ca(OH)_2$, $Mg(OH)_2$, SiO_2, NaCl, Na_2SO_4, $Na_2O \cdot Al_2O_3 \cdot 4SiO_2 \cdot 2H_2O$, $CaO \cdot SiO_2 \cdot H_2O$, Cu, S, CaS, and $Ca_3(PO_4)_2$—*T. E. Larson.*

Zeiss-Pulfrich Apparatus for Measuring Turbidity. Department of Water Control (French). Mangerel and Coin. Ann. Serv. tech. Hyg. (Paris) ('37). Describes design of apparatus which combines a Zeiss turbidimeter

and a Pulfrich photometer and which can be used to determine the turbidity of water in terms of glass standards. Tables given showing the relation between standard silica suspensions, glass standards used to determine comparative turbidities and a special standard, the absolute turbidity of which had been determined by optical methods.—*W. P. R.*

Preparation of "Optically-empty" Distilled Water. Department of Water Control (French). VILLEMAINE. Ann. Serv. tech. Hyg. (Paris) ('37). Completely clear water, for the preparation of suspensions of standard turbidity, is prepared by distilling water from a cylindrical copper vessel in which an electrical heater is suspended to a depth 12 cm. above the bottom of the vessel. Any solid particles which separate fall to the bottom and are not carried over in the steam.—*W. P. R.*

A Compact Field Laboratory for Sanitary Chemistry. JAMES G. WEART. Ind. Eng. Chem.-Anal. Ed. **11:** 43 (Jan. '39). Description of field laboratory kit used by sanitary engineers of Dept. of Public Health and State Sanitary Water Board of Illinois in supervising water and sewage treatment plants. Kit weighs 30 lbs. and is substantially built of ½" birch with acid- and alkali-resistant finish inside and out. Inside of lid, covered with white celluloid, forms work table, being supported by removable metal rods screwed into outside corners. Buret support screws into inside of lid. Three 10 ml. serological pipets, closed with rubber tube and bead at top, and used as burets, are held by clips in removable tray when traveling. Kit provides for detg. pH between 4.0 and 11.0 in steps of 0.2, and also has wide range indicator. Also included are reagents and equipment for detg. dissolved oxygen by Winkler method, alkalinity and soap hardness by Standard Methods, residual chlorine with standards prepd. from Scott's buffered chromate-dichromate solns., carbon dioxide by calculation from alkalinity and pH using slide rule or log table, and temperature. Reagents are carried in 1 oz. or 2 oz. bottles, and dissolved oxygen detns. are made in 4 oz. glass stoppered bottles. Graduate and calibrated bottle are used for measuring samples for alkalinity and soap hardness. Distilled water is available for diluting samples. Extra capacity is available to adapt kit readily to test for iron, nitrite, copper and ammonia. Kits were prepd. by Rascher and Betzold for $90 each in lots of 10.—*Selma Gottlieb.*

The Influence of Air on High Pressure Steam Sterilization. O. MÜNDEL. Zent. f. Bakt., I. Abt. Orig. **143:** 469 ('39). As steam enters the sterilizing chamber, the air is displaced, but in the case of close-packing with, say, textiles, "islands" or "pockets" of air are formed. These make access of steam more difficult, so that the heating-up process is retarded. In an empty chamber the heating is rapid and regular in spite of a residue of air. If the autoclave is evacuated first, then the steam penetrates the most inaccessible spaces very quickly. The germicidal power of steam is not altered by the presence of air; the only difference observed is a longer time taken for complete sterilization. Experiments were carried out using well-manured earth spread on trays, placed in the folds of cloth, or in tubes of broth medium. These were placed on shelves situated at different levels in the autoclave. The author gives a simple formula for the calculation of the residual air present. The

mixture of air and steam in the autoclave follows Dalton's Law of Partial Pressures and Berthollet's Law of Diffusion so that,

$$R = \frac{(p - p_i)(273 + t)\ 100}{273 + t_i}$$

where R = percentage residue of air,
t = room temperature,
p = pressure in chamber,
p_i = pressure of saturated steam at t_i,
t_i = temperature in chamber.

—*B. H.*

METERS AND SERVICES

It Pays to Test Meters. A. P. KURANZ. W. W. Eng. **92:** 1114 (Aug. 30, '39). Waukesha, Wis., (pop. 18,000) water supply derived from three 1783′ to 1918′ deep wells, each with av. capac. over 24 hrs. of 1070 g.p.m.—total 4.6 m.g.d. Meters considered important part of problem in efficient operation of utility. In av. 3 bedroom house with water pressure of 40 lbs. or more, a $\frac{3}{4}$″ service found ample; where pressure below 40 lb. one-inch size gives better service. Recent check on 229—$\frac{5}{8}$″ meters brought to shop disclosed av. months since last tested, repaired or set as new was 73.5 mo., the youngest being 11 mo. and oldest 88 mo. Only 7 showed no registration on $\frac{1}{4}$ g.p.m., 10 under 80% accuracy on same flow, 11 between 80 and 84.9%, 19 between 85–89.9%, 26 from 90 to 92.9% and 156 from 93 to 102.1%. Av. water measured was 262,379 gal., av. cost of repairs per meter to bring meter to 95% accuracy or better on $\frac{1}{4}$ g.p.m. was 24.2¢ for materials only. No meter now left in service more than 84 mo. nor allowed to measure more than 600,000 gal. Believed testing meters honestly results in decided improvement in revenue.—*Martin E. Flentje.*

New Van Nuys Meter Shop Serves Valley. ANON. The Intake (Los Angeles Dept. of Water & Power) **15:** 12: 6 (Dec. '38). To test and repair approx. 25,000 domestic and 3600 intermittent irrigation meters in San Fernando Valley area of dep't. territory, new 47′ x 71′, 2 story shop built at Van Nuys, Calif. Work and storage benches are of concrete, this having proven superior to wood or metal in yrs. of service in other shops of dept. Shop contains 2 standard testing units, 1 for meters up to 1″, other for meters up to 2″, both units with tanks of 10 cu. ft. capac. Meters larger than 2″ tested with water from a 100 cu. ft. concrete tank. Other equipment includes motor driven buffer, lathes, drill press and other shop units.—*Martin E. Flentje.*

Water Meters in Indianapolis. W. W. DEBERARD. Eng. News-Rec. **121:** 593 (Nov. 10, '38). Practice of Indianapolis Water Co. in testing, selection and maintenance of meters is reviewed. In all, company owns, installs and maintains 76,000 meters. Per capita consumption, based upon amount of water pumped, is 75 g.p.d. Meters read monthly. Particular attention paid

to accuracy on low flows. Customers' meters register 85% of water pumped, 10% is attributed to city and company uses, distr. system leakage and meter under-registration, and 5% is unaccounted for. Studies have disclosed that no plumbing system is bottle-tight and that bulk of domestic use is at rate under 5 g.p.m. All $\frac{5}{8}$″ meters brought in for complete test when registration reaches 100,000 cu. ft., and larger meters at proportionately higher registrations. Replacement of meters that are inaccurate on low flows has been found advantageous. About 2000 meters are retired each yr., i.e., when cost of repairs and parts exceeds 50% of original cost. Meter readers are specially trained in deportment. Chart included showing results of accuracy tests at various rates on 155 meters of 5 different makes after passing 100,000 cu. ft. —*R. E. Thompson.*

Maintenance Methods—Practices Employed at Pittsburgh, Pa. NATHAN N. WOLPERT. W. W. Eng. **92:** 70 (Jan. 18, '39). Officials of Pittsburgh's Bureau of Water include James H. Kennon Managing Engr. and Fred J. Deuschle, Gen'l Service Foreman. Pop. of city—670,000. Bureau's practice to remove each domestic meter every 7 yrs. unless defective sooner. Meter rebuilt, dials re-enamelled or replaced, gear trains cleaned or renewed and casing painted. Consumer pays for service and buys meter, any meter of $\frac{5}{8}$″, $\frac{3}{4}$″, or 1″ size bought from firms having contracts with city, larger may be from any firm on approved list. Meters tested on full, $\frac{1}{4}$″, $\frac{1}{16}$″ and $\frac{1}{32}$″ flows, must not have error over 2%. One man can rebuild four $\frac{5}{8}$″ to 1″ meters in 1 day at av. cost of $1.11 per meter; 5987 repaired and tested in '37. 40–45% of consumers on flat rate. Meters read and billed quarterly. Pipe joints on ordinary runs may be of compound or lead. Four ft. pipe cover ample protection against frost. For past few yrs. cement lined centrifugal cast iron pipe used up to 24″ size, above that pipe is sand cast and lined; 12′ lengths used. Hydrants painted yellow, inspected 3–4 times per yr. Provisions against flood dangers described.—*Martin E. Flentje.*

Effects of Aluminum and of Antimony on Certain Properties of Cast Red Brass. HAROLD B. GARDNER AND CHARLES M. SAEGER, JR. J. Research Nat. Bur. Standards **22:** 707 (Jun. '39). Additions of aluminum in amounts from 0.005 to 0.10% were made to red brass, made of remelted metal, of the nominal composition of 85% of copper and 5% each of tin, zinc, and lead. Determinations of tensile properties, density, hardness, the running property, and the ability to withstand hydraulic pressure, made on various types of test bars and test specimens, showed that all additions of aluminum had a deleterious effect on the properties and the structure. The effect was more pronounced with the higher concentrations of aluminum and was particularly evident on the ductility and the resistance to hydraulic pressure. Additions of antimony to this alloy in the amounts ranging from 0.10 to 0.25% had only a nominal effect on all of the properties studied except the running property, which was increased. These results indicated that the presence of 0.25% of antimony is permissible, but that even 0.005% of aluminum has a deleterious effect on the alloy.—*Abstract by the Bureau.*

Choosing Meters for Important Services. HAROLD W. GRISWOLD. W. W. and Sew. **86:** 263 (Jul. '39). At Hartford, Conn., an automatic rate of flow recorder is used to facilitate determination of meter size. Three cases are described in which the recorder was used to check rates of flow on comparative installations of battery meters and compound meters. The compound and battery meters were connected in series. More accurate registration was obtained from the compound meters.—*H. E. Hudson, Jr.*

Water Services, *Domestic.* ANON. W. W. Inf. Exch., Canadian Sect. A. W. W. A. **3:** C: 5: 15 (May '39). Tabulation of Canadian municipalities showing no. of domestic services, no. of various sizes, most common size and percentage that no. of latter is of total. Most common size is ½″ in 27 communities, ¾ and ⅝″ in 1, ⅝″ in 25, ¾″ in 16 and 1″ in 1. *Commercial. Ibid.* **3:** C: 6: 19. Data, as above, are given regarding commerical and industrial services. Most common size is ½″ in 13 municipalities, ⅝″ in 15, ¾″ in 10, 1″ in 8, 1¼″ in 1, 1½″ in 3, 2″ in 2, and 3, 4 and 6″ in 1 each. *Summary. Ibid.* **3:** C: 7: 24. Tabulation showing no. of services (domestic, industrial and total), no. of services not in use, population per service and services per mi. of main. Max., min. and av. population per service is 13.7, 0.3 and 4.2, respectively, and max. min. and av. no. of services per mi. of main 250, 40 and 111, respectively.—*R. E. Thompson.*

Location of Water Meters. ANON. W. W. Inf. Exch., Canadian Sect. A. W. W. A. **3:** C: 4: 13 (Mar. '39). Practice in Canada in regard to location of meters is shown in tabulation of 65 municipalities. Majority are located in basements.—*R. E. Thompson.*

How's Your Meter Setting? D. R. TAYLOR. W. W. & Sew. **86:** 201 (May '39). Description of concrete meter vault which may be made by water plant personnel. Cost is $2.10.—*H. E. Hudson, Jr.*

Putting the Water Works G-Man to Work. ROGER W. ESTY. W. W. and Sew. **85:** 839 (Sep. '38). The "G-Man" is a recording meter used for checking the flow through a water service to aid in selecting meter size. Also used to account for unusual consumption. Applications are described.—*H. E. Hudson, Jr.*

Investigation of House Installation for Leakage and Correct Size of Pipe. EDUARD BAESE. Gas-u. Wasser. **82:** 402 (May, 27, '39). The waterworks is interested in house installation to avoid waste, water hammer, back siphoning of dirty water, damage to meters and back flow of hot water. Of 200,000 installations about 40% were found to have leaks with up to 20% of the total consumption. Leaks are found with an apparatus that registers the meter reading on a drum. It is normally set to run on a 24-hr. basis, but can be used also with a 10 times larger sensitivity for a 1 hr. period. It is set on a regular meter and can be used to check the meter. The correct size of the pipe is checked in conjunction with the above apparatus by also using the following apparatus: (a) a high sensitive recording apparatus for the momentary use of water, (b)

pressure recording apparatus set at the meter and in the ends of the lines in the house. Pressure drops in these ends are larger and a drop below 1 kg. per sq. cm. (14.2 lbs. per sq. in.) is considered as an indication of too small pipes in the installation.—*Max Suter.*

Tellurium Lead Pipes and Sheets. ANON. Wtr. and Wtr. Eng. (Br.) **41:** 257 (May '39). Among the advantages of tellurium lead over ordinary lead may be included: increasing resistance to grain growths under high temperature, avoiding burnt joints; quality control is more efficient; latent strength is higher; mechanical strength is greater; there is higher resistance to bursting by frost action; there is greater resistance to metallic fatigue; and increased resistance to corrosion. Tensile strengths of 3,000 to 3,500 lb. per sq. in., with 75% elongation in 8", can be attained.—*H. E. Babbitt.*

The Solution of Lead in Drinking Water. H. HAUPT. Gesundheits.-Ing. **62:** 161 ('39). Discusses the mechanism whereby waters containing dissolved oxygen and carbon dioxide dissolve lead from lead pipes, and suggests means to reduce the danger of lead poisoning. In older houses with lead pipes a change in quality of water supply can cause solution of the protective coating which usually forms on the pipe, so leading to new outbreaks of poisoning. Distilled water, saturated with air, will dissolve 85 mg. of lead per liter or, if saturated with carbon dioxide, it will dissolve 16 mg. per liter. Water of this kind collected in rain-water cisterns, should never flow through lead pipes. If oxygen alone is present, lead hydroxide is formed; with small amounts of carbon dioxide, a basic lead carbonate forms (solubility 0.4 mg. per liter) which, with calcium carbonate and iron oxide, forms a dense protective coating, preventing further solution of lead. As the concentration of free carbon dioxide rises, lead carbonate (solubility 1.1 mg. per liter) or bicarbonate (easily soluble) are formed. Waters with hardness greater than 7° (German) form coatings within a few weeks. Soft waters containing iron seldom form protective coatings; the iron oxide is apparently unable to adhere to the smooth walls of the lead pipe. Until a protective coating has formed on new pipes, water which has been standing in the pipes should be run to waste. Plants for removal of acidity and for adding lime to soft waters should be inspected regularly to prevent the entry of untreated water in the distribution system where previously formed coatings would be dissolved. In lead pipes treated with sulfur compounds, lead sulfide may be oxidized to soluble lead sulfate and the process therefore does not give adequate protection. Tin-lined lead pipes are liable to electrolytic corrosion which uncovers the lead. —*W. P. R.*

Corrosion of the Water Supply Pipes in Apartment Houses. GERARD RICHARD. Tech. Sanit. Munic. (Fr.) **34:** 54 (Apr. '39). Corrosion is most troublesome in pipes which are concealed in walls and floors. May be caused by nature of the water itself. Water from soil derived from acidic rocks has a hardness of from 3 to 10 French degrees and an acid reaction; water from regions of basic rocks contains sulfate and lime carbonates, has a hardness of from 20 to 120 degrees and a neutral reaction; water from regions near the sea contains

chlorides and is generally corrosive; water from swampy regions contains traces of organic acids which often give the water unexpected corrosive properties. These facts concerning different waters determine the kind of metal to be used for water pipes. Corrosion may be induced by minute cavities in the inner surfaces of the pipes, since these cavities or fissures are less aerated by the air in suspension in the liquid; this results in a process called "differential aeration." Impurities in the metal used may lead to corrosion. Another source of corrosion is stray electric currents which originate in faulty insulation of the electric supply system. Stray currents from power installations which are used continuously may result in corrosion to a dangerous degree. Alternating current should give less trouble than direct current. This form of corrosion has been encountered frequently in many new apartment houses. A survey in the departments of Seine, Sein-et-Oise and Seine-et-Marne showed that the water pipes were constantly charged with a negative potential. Following suggestions made as means of combating stray currents: (1) protect the outside of water mains; (2) drain the soil surrounding the pipes; (3) avoid acidification of the soil by vigorous sanitary regulations; (4) place joints and couplings in the vertical sections of pipes; (5) avoid porous material in the construction of foundations; (6) insulate pipes within walls and use couplings with weak electric resistance.—*Willem Rudolfs.*

Electrical Anti-freeze Device for Water Pipes. N. V. Holl. Water (Netherlands) **23:** 19 (Jan. 27, '39). A small cable is used which is wound spirally around the pipe to be protected, about 4″ between turns. The core of the cable is a resistance wire which is embedded in special heat-resistant insulation "Ozuriet," the whole being enclosed in a tightly fitting lead sheath. The usual voltage in Holland is 220 and the resistance wires are calibrated as to composition and diameter so that a definite length of wire under this voltage will attain the same temperature. Five grades of wire are available, the length in each case being such that when freely suspended in the air under 220 volts its temperature will be about 70°C. (the wire of course being warmer). The lengths thus vary from 62′ to 253′ and are in each case the shortest allowable. Longer lengths may be used, but the temperature will be correspondingly lower. Indoor pipes are finished off with a covering of asbestos paper sufficient to retain the heat. Outdoor pipes are given a thin layer of asbestos or magnesia insulation and over that a coating of "Ozuriet." If desired, an automatic device can be added which will switch on the current whenever the water falls below a specified temperature.—*Frank Hannan.*

Electrical Pipe Thawing. F. J. Hirner. W. W. and Sew. **85:** 1116 (Dec. '38). Instructions and rules covering electrical pipe thawing. Principle of the method, kind of generator, procedure, amperages, times of thawing, are covered briefly, as well as cautions to be observed.—*H. E. Hudson, Jr.*

HYDROLOGY

The Unit Hydrograph Principle Applied to Small Watersheds. E. F. Brater. Proc. A.S.C.E. **65:** 1191 (Sep. '39). The paper demonstrates that the unit hydrograph principle may be applied to small watersheds and that the dis-

tribution graphs and pluvigraphs are valuable tools for the analysis of surface run-off. It further indicates that the unit hydrograph method is one of the best practical devices for predicting flood flows. It is shown that the ground-water contribution may be separated from the surface run-off with considerable assurance in the case of hydrographs resulting from isolated rains. The unit hydrograph is an important aid in the separation of ground-water contribution from surface run-off in the case of a complex hydrograph produced by a series of rains. Any rain that is sufficiently intense to produce surface run-off will produce a unit hydrograph if its duration is equal to, or less than, the period of rise. The period of rise is a function of the watershed characteristics and is the time required for the major portion of the watershed to release its accumulated rainfall load. There is a definite correlation between the shape of the distribution graphs and the watershed characteristics. The reproduction of known flood hydrographs by the use of the pluvigraph provides a method of determining the variation of run-off coefficient or infiltration capacity on a watershed throughout the period of surface run-off. The unit hydrograph principle provides a fundamentally sound method of predicting run-off.—*H. E. Babbitt.*

Run-off Distribution Graphs from Precipitation Occurring in More Than One Time Unit. WILLIAM T. COLLINS. Civ. Eng. **9:** 559 (Sep. '39). When distribution graph coefficients are to be determined the first factor that must be considered is the time unit. Actual computation of the distribution coefficients is by a systematic trial-and-error method. After the time interval has been selected the average precipitation for each interval is listed. The total amount of surface run-off is estimated from the hydrograph, by subtracting a reasonable base flow, and assigning the remainder to the various intervals of rain. After the inches of run-off for each period of rain have been determined they are converted to c.f.s. per time unit. A distribution graph for the drainage area is estimated and used on all of them except the largest. Coefficients from this distribution are compared with the original estimated and if there is little difference between them they are used as found. A sample computation of a set of distribution coefficients is shown.—*H. E. Babbitt.*

Analysis of Run-off Characteristics. (*Discussion of previous paper*, see abstract J.A.W.W.A. **31:** 1246 (Jul. '39)). Proc. A.S.C.E. **65:** 1045 (Jun. '39). C. S. JARVIS. Retention of rainfall by surface storage and induced infiltration on a representative portion of the drainage basin area, such as one-fourth or one-half, with no escape to drainage channels by overland or other surface flow, must inevitably reduce the total volume of run-off in about the same proportion. Reduction of run-off from similar areas by reason of detention works, increased vegetation, lengthened course of travel, reduced gradients as well as velocities of flow, and prolonged contact of such water with the soil surface, account for a proportionate decrease of run-off volume, and if the treated areas are widely distributed throughout the drainage basin, the base of the flood hydrograph could not conceivably be shortened as a result of such retention and retardation through conservation measures. HOWARD M. TURNER: The discussor does not believe that the basic hydrograph given is

one which will be produced on a river by a uniform rainstorm just equalling the time of concentration, particularly in respect to the sharp break at the top where, in the author's description, the concentration curve changes immediately to the storage curve. This assumes that the stopping of the rainfall all over the watershed is felt to its full extent immediately at the point of measurement.—*H. E. Babbitt.*

The Trend of Annual Rainfall in Scotland. A. H. R. GOLDIE AND H. E. CARTER. Surveyor (Br.) **95:** 733 (Jun. 2, '39). *Presented before the Inst. of Water Engineers.* In West Scotland there has been a general rise of rainfall during the last 45 years which has no counterpart in East Scotland. The rise in the western districts has been from 89% of "normal" in 1896 to 116% in the decade ending in '32. Over Scotland as a whole the mean rainfall of the very dry year, 1887, represented 80% of the average, while in the wet year, '03, the figure was 129%. It is not surprising, therefore, that communities that have found their supplies entirely adequate may be placed in a difficult position if and when there is a return to the rainfall conditions of 1880–1900. *Discussion. Ibid.* p. 752 (Jun. 9, '39). W. N. MCLEAN: Showed records of the Kilmarnock gage which corroborated author's findings. W. DUNBAR (Kilmarnock): Pointed out that a 200-yr. period is required to bring out cyclic tendencies, none of which is shown in the author's paper.—*H. E. Babbitt.*

The Duties and Achievements of the Reich Hydro-Meteorological Service. D. W. FRIEDRICH. Deut. Wasserwirtschaft **34:** 312 (Jul. '39). Climate in Germany is fairly uniform and precipitation is tolerably evenly distributed throughout the year. Taking precipitation for the normal year to be 100, the yearly figures lie within 60 and 140. Monthly variations are, of course, much wider. At all seasons of the year there occur rainless, or almost rainless, months, and also months with precipitation double, and even treble, the normal. Fluctuations such as these impose responsibilities upon the water control boards, seeing that the soil of the Reich has to feed a population of 140 persons per square kilometer, a highly developed industry has to be maintained, and the manifold evils of flood and drought have to be warded off as far as possible. It is obvious that for the control boards to function properly, they must have available an immense body of data covering every corner of the Reich. One of the principal duties of this Service is to compile exhaustively all basic data and hold them at the disposal of the control boards as may be required. Thus, catchment areas have been measured, continuous observations of water levels and stream flows, of ground water levels and of spring flows, have been maintained, normal values in every case determined, and abnormalities satisfactorily accounted for. Although the Reich weather service in its present form was organized only in '34, records for the more important phenomena extend back over many decades. Rainfall observation stations when the Service was formed numbered upwards of 4000; their number is now in excess of 7000. Monthly and annual rainfall charts are published in color, making the planimetric computation of total rainfall very easy. Flood warnings also published. Special instruments have been devised for rainfall

and snowfall measurement in difficultly accessible mountain regions. The recording rainfall meters have already been described by Reinhold (Deut. Wasserwirtschaft **34**: 275 (Jun. '39)). Bibliography of the most authoritative publications dealing with Reich rainfall statistics is given. Pointed out that, if a sufficiently great number of years be taken, the mean annual rainfall must be equal to the mean annual run-off plus the mean annual evaporation, as the other factors tend to cancel out. Rate of evaporation is governed chiefly by the atmospheric temperature. Methods have been proposed by which to deduce the run-off in cases where precipitation and mean annual temperature are known; but approximations thus obtained would be very rough. Fischer has devised a statistical method, making certain simplifying assumptions, by which, from observations of rainfall and run-off, to deduce for each half-year the local evaporation and the addition to ground water reserve. Investigation, including comparison with the lysimeter, is proceeding. The mean annual evaporation in the valleys of the principal rivers of the Reich is about 450 mm. At the Eberswalde lysimeter, by varying the nature of the soil, and of the vegetation, and of the height of the water table, values varying from 178 to 706 mm. have been obtained. From open water, values obtained at different sites in the Reich varied from about 600 to about 950 mm. ★[A lysimeter is a device for measuring the percolation of water through soils. No description of a lysimeter is given in the article. The Eberswalde lysimeter is evidently an unusual type of instrument. It serves as an evaporation gage because the evaporation equals precipitation minus percolation.] ★—*Frank Hannan.*

Standardizing the Dimensioning of Sewerage Systems. FRIEDRICH REINHOLD. Deut. Wasserwirtschaft **34**: 275 (Jun. '39). This paper deals chiefly with rainfall and is most authoritative. For many years past a large number of German cities have kept automatic rainfall records. Only a few of these were worked up to obtain rainfall characteristics and cooperation was absent. The advantages of a central clearing-house for all these valuable statistics and of uniform procedure in working them up were overwhelming and obvious. All concerned heartily concurred and Dr. Reinhold was put in charge. A map of Germany is given showing the locations of 525 automatic recorders, including those of the Prussian Government, but excluding those of the Reich, substantially as in '34. After taking stock of the situation, Dr. Reinhold caused two important official papers to be issued in '36, the first (ADN 1936), to insure correct adjustment and servicing of the instruments and, therefore, accuracy, and the second (AAR 1936), on the method to be followed in working up the records, so that data from all stations should be strictly comparable. The laborious task of working up on a uniform basis all existing records is even yet not complete, but progress has been so great that a nomograph (reproduced) incorporating an outline map of the Reich, with certain fundamental data, has been constructed from which it is possible to read off instantaneously (and with approximate accuracy) the intensity, r, in liters per second per hectare, of a rainfall for any place within the Reich, of any given duration, T, in minutes, and any given frequency, n, per year. This nomograph also gives the envelope of the rT lines corresponding to the heaviest rainfalls on record in

Germany, the intensities and corresponding durations of which can therefore be at once read off. The principles upon which the relations were obtained which are expressed in the nomograph are given. By comparison of many data from different localities widely scattered, certain general rules were found to hold all over the Reich. Factors other than rainfall are also touched upon, but are not of water works interest.—*Frank Hannan.*

Water Control and Agriculture. WILHELM FRICKMANN. Deut. Wasserwirtschaft **34:** 263 (Jun. '39). Non-ligneous plants are 80% water, and the consumption of water by the vegetable world is immensely greater than any other. Thus, the annual crop on the 29 million hectares of agricultural land in Germany is estimated to need 125 billion cubic meters of water, while the people themselves use less than 8 billions. Deficiency in water supply at the critical period of growth means harvest failure; excess water, on the other hand, is also undesirable. It is estimated that 7.5 million hectares of the agricultural area needs draining, while 3.5 million hectares need irrigation and a further 6.5 million hectares would be improved by irrigation. In view of the shortage, not only of food, but of vegetable fibers, etc., the importance of this problem is very great.—*Frank Hannan.*

Preliminary Hydrographic Report for 1938 (Italy). Issued by Italian Hydrographic Office. Ann. Lavori Pub. (Rome) **77:** 474 (May '39). Report covers atmospheric temperatures and rainfall at 17 of principal cities; stream-flow at 22 stations on 21 of the principal rivers; and amount of solid matter in form of turbidity carried down by 8 of these rivers. Data drawn up with much skill in eight tables to include comparisons with preexisting records, which reach back in some few instances to the first decade of the nineteenth century. Notable features of the year are briefly discussed and 24 graphs bring home forcibly certain salient points.—*Frank Hannan.*

Hydrology and Water Supply of Jaffna, Ceylon. S. MAHADEVA. Wtr. and Wtr. Eng. (Br.) **41:** 250 (May '39). Jaffna is a peninsula at the north end of the island of Ceylon, within the equatorial region. Av. annual rainfall over the peninsula ranges from 46.5″ to 56.5″, the mean being 50.8″. Droughts occur every 3 or 4 years, the worst drought occurring in '18 when the rainfall for 7 mos. was 2.33″ followed in Nov. of the same year by a precipitation of 31.43″. The geological conditions are such that the soil maintains a high ground water table even in dry periods and fresh water is found close to the sea coast. Kulams, or water lanes, abound, and serve to prevent minor floods and diminish the destructiveness of large floods. This was clearly demonstrated in Nov. '18. Cavernous formations exist underground sometimes causing disappearance of water from kulams, accompanied by whirlpools and suction. The Puttar well is in such a region. It is historically and geologically famous. Most water supplies are from wells drawing from large subterranean reservoirs.—*H. E. Babbitt.*